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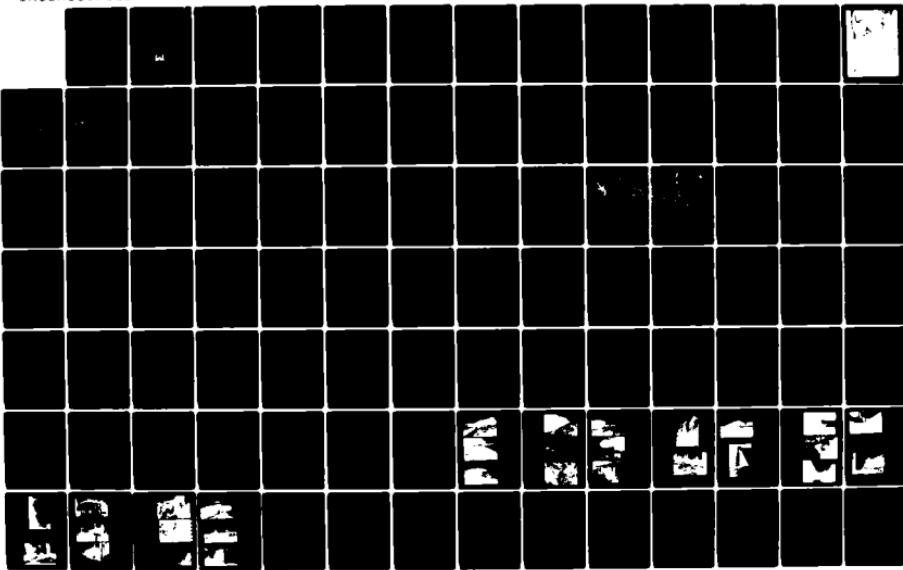
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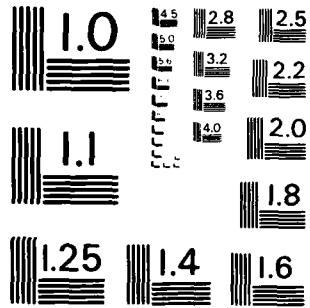
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AD-A145 564

CONNECTICUT RIVER BASIN,  
RUSSELL, MASSACHUSETTS

COBBLE MOUNTAIN RESERVOIR DAM  
MA 00068

PHASE I INSPECTION REPORT  
NATIONAL DAM INSPECTION PROGRAM

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DEPARTMENT OF THE ARMY  
NEW ENGLAND DIVISION, CORPS OF ENGINEERS  
WALTHAM, MASS. 02154

MARCH 1980

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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
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4. TITLE (and Subtitle) Cobble Mountain Reservoir Dam		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE March 1980
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19. KEY WORDS (Continue on reverse side if necessary and identify by block number) <b>DAMS, INSPECTION, DAM SAFETY,</b> Connecticut River Basin Russell, Massachusetts		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The dam is a sluiced earth fill between massive rock toe embankments. The top of the dam is about 730 feet long and about 250 feet above the valley bottom at the toe. The dam is generally in GOOD condition, well maintained, with no excessive or dangerous leakage. The preliminary hydrologic and hydraulic analysis for this LARGE size, HIGH hazard dam indicated that the spillway is adequate. The test filled is the PMF and for a tributary drainage area of 45.8 square miles of mountainous terrain which is more than 80% forested, the unit discharge rate is 440 CFS/square miles.		



DEPARTMENT OF THE ARMY  
NEW ENGLAND DIVISION, CORPS OF ENGINEERS  
424 TRAPELO ROAD  
WALTHAM, MASSACHUSETTS 02254

REPLY TO  
ATTENTION OF:

NEDED

DEC 9 1980

Honorable Edward J. King  
Governor of the Commonwealth of  
Massachusetts  
State House  
Boston, Massachusetts 02133

Dear Governor King:

Inclosed is a copy of the Cobble Mountain Reservoir Dam (MA-00068) Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Quality Engineering, the cooperating agency for the Commonwealth of Massachusetts. In addition, a copy of the report has also been furnished the owner, City of Springfield Water Department, Mass.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Quality Engineering for your cooperation in carrying out this program.

Sincerely,

WILLIAM E. HODGSON, JR.  
Colonel, Corps of Engineers  
Acting Division Engineer

Incl  
As stated

COBBLE MOUNTAIN RESERVOIR DAM

MA 00068

CONNECTICUT RIVER BASIN  
RUSSELL, MASSACHUSETTS

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PHASE I INSPECTION REPORT  
NATIONAL DAM INSPECTION PROGRAM



NATIONAL DAM INSPECTION PROGRAM  
PHASE I INSPECTION REPORT

Identification No.: MA 00068  
Mass. DPW No.: 1-7-256-11  
Name of Dam: Cobble Mountain Reservoir Dam  
Town: Russell  
County and State: Hampden County, Massachusetts  
Stream: Little River  
Date of Inspection: November 15, 1979

BRIEF ASSESSMENT

The Cobble Mountain Reservoir Dam embankment is located in Russell, Massachusetts on the Little River about thirteen (13) miles above its point of discharge into the Westfield River at a point that is about nine miles above its confluence with the Connecticut River. This facility is located where the three Towns of Blandford, Granville and Russell abut, with the diversion tunnel inlet and the upstream toe dam lying in Blandford and the emergency spillway and intake control tower located in Granville. The dam is a sluiced earth fill between massive rock toe embankments. The top of the dam is about 730 feet long and about 250 feet above the valley bottom at the toe. A spillway with a reinforced concrete overflow weir 135 feet long standing six (6) feet above the spillway floor has been cut through rock about 1500 feet south of the dam. An intake and power tunnel 10 feet by 11.5 feet has been driven through rock about 3000 feet south of the dam. A diversion tunnel 10.5 feet by 10.5 feet is located through rock about 1000 feet northwest of the dam.

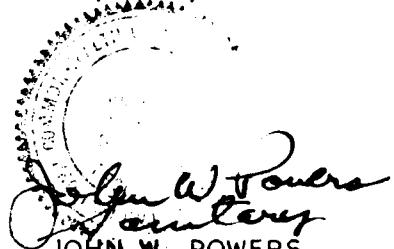
The dam is generally in GOOD condition, well maintained, with no excessive or dangerous leakage. However, the stability of the embankment in modern engineering terms, particularly with respect to seismic loading and the potential for liquification of the hydraulic fill has not been analyzed, and for this reason, the dam has been rated as FAIR.

The preliminary hydrologic and hydraulic analysis for this LARGE size, HIGH hazard dam indicates that the spillway is adequate. The test flood is the Probable Maximum Flood and for a tributary drainage area of 45.8 square miles of mountainous terrain which is more than 80% forested, the unit discharge rate is 1440 CFS/square mile. The inflow to the reservoir from the maximum probable flood would be about 66,000 cfs. When routed through the reservoir by preliminary methods, the maximum outflow would be about 38,000 cfs which would flow over the spillway weir with about 15.3 feet of head. This leaves about 4.7 feet of freeboard to the top of the dam.

The spillway has a capacity of approximately 145% of the routed test flood outflow.

Failure of the dam could cause floodwater to race down the Little River valley as much as 90 feet deep. The City of Westfield could be flooded up to about elevation 168 ft. MSL. This is about twenty (20) feet above Main Street and Broad Street and would reach nearly to Noble Hospital. About 5,000 acres of Westfield and Southwick might be flooded. Dikes along the Westfield River could be topped by several feet and much of the Merrick area of West Springfield and the Eastern States Exposition grounds could be flooded as well as much low land in Agawam along the Westfield and Connecticut Rivers.

The recommendation for seismic stability and liquification analysis should be initiated within one (1) year of receipt of this report by the Owner. The remedial measures listed in Section 7 should be implemented within two (2) years of the receipt of this report by the Owner.



John W. Powers  
Powers Engineering  
JOHN W. POWERS  
MASSACHUSETTS REGISTRATION 23106

This Phase I Inspection Report on Cobble Mountain Reservoir Dam (MA-00068) has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Armand Martesian

ARAMAST MARTESLIAN, MEMBER  
Geotechnical Engineering Branch  
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, MEMBER  
Design Branch  
Engineering Division

Richard J. DiBuono

RICHARD DIBUONO, CHAIRMAN  
Water Control Branch  
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Lyle  
JOE B. FRYAR  
Chief, Engineering Division

## PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

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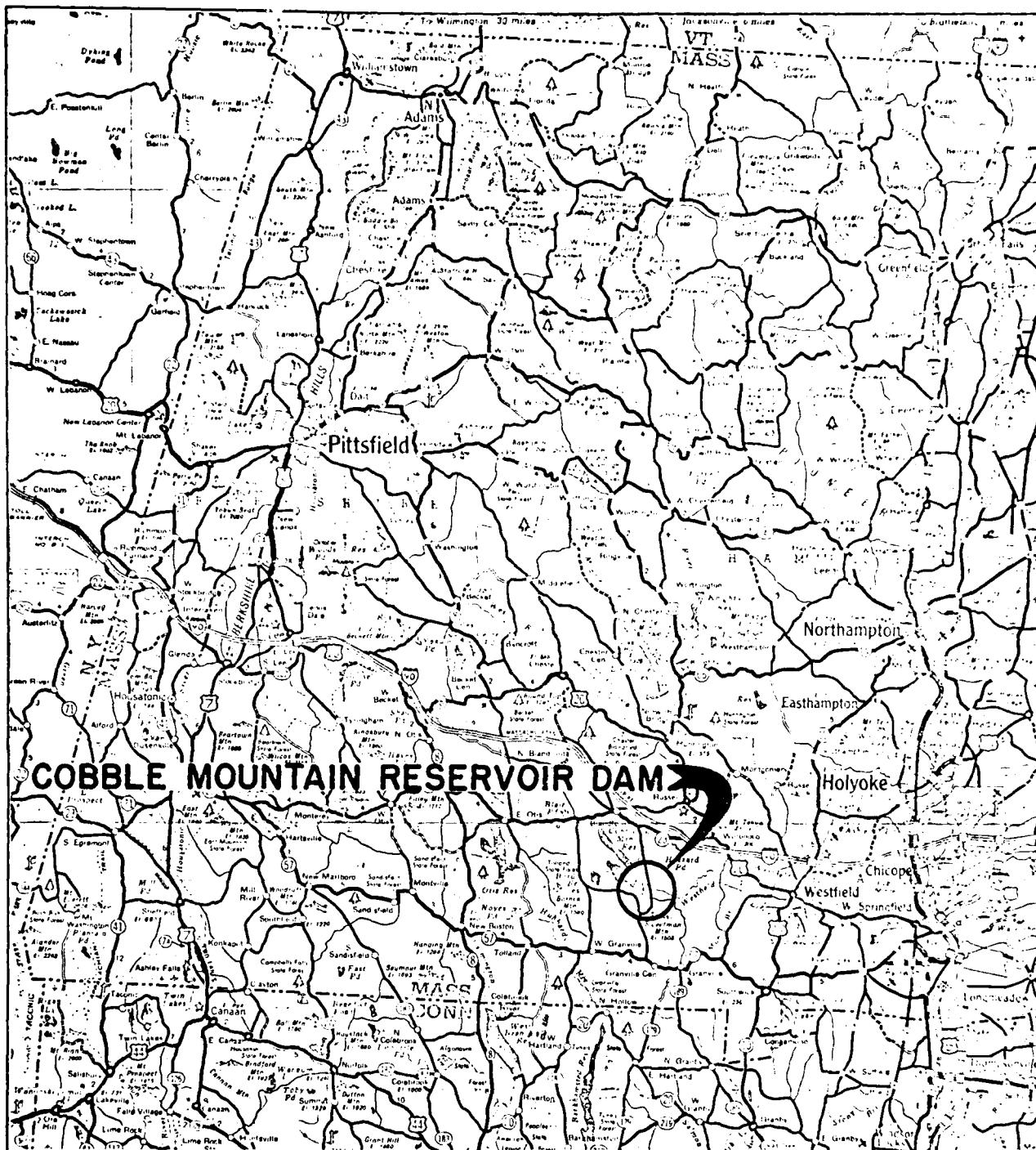
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TIGHE & BOND / SCI  
CONSULTING ENGINEERS  
EASTHAMPTON, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

### NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

### LOCUS PLAN I

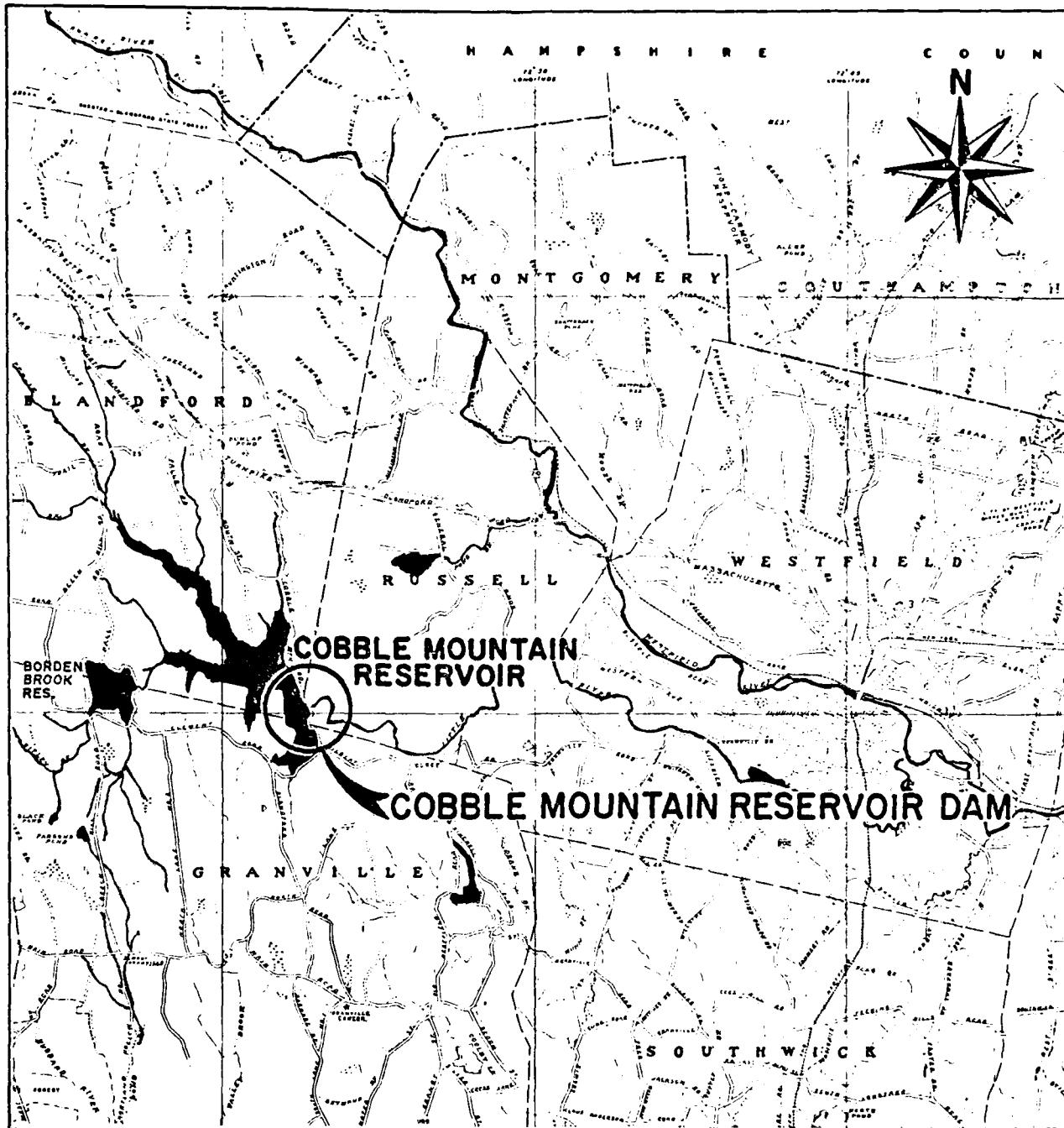
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HAMPDEN COUNTY MASSACHUSETTS

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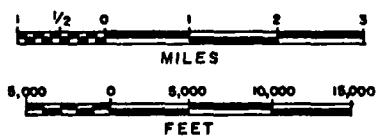
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TIGHE & BOND / SCI  
CONSULTING ENGINEERS  
EASTHAMPTON, MASS.

U.S.ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN 2

COBBLE MOUNTAIN RESERVOIR DAM (MA 00068) RUSSELL  
HAMPDEN COUNTY MASSACHUSETTS

SCALE: AS NOTED  
DATE: MARCH 1980

PHASE I INSPECTION REPORT

COBBLE MOUNTAIN RESERVOIR DAM

SECTION 1

PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Tighe & Bond/SCI has been retained by the New England Division to inspect and report on selected dams in Massachusetts. Authorization and notice to proceed were issued to Tighe & Bond/SCI under a letter of October 24, 1979 from Colonel William E. Hodgson, Jr., Corps of Engineers. Contract No. DACW 33-80-C-0005 has been assigned by the Corps of Engineers for this work.

(b) Purpose

- 1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.
- 2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-federal dams.
- 3) Update, verify, and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-federal dams in the high hazard potential category based upon location of the dams, and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

Cobble Mountain Reservoir Dam is located on Little River about thirteen (13) miles from its confluence with the Westfield River which is at the east end of the City of Westfield. Cobble

Mountain Dam is west of the City of Westfield about 8.9 miles from Elm Street and Main Street via Granville Road and Wildcat Road in the Town of Russell. The spillway and intake tunnel are in the Town of Granville. The dam is shown on U.S.G.S. topographic quadrangle "Blandford" and the spillway on U.S.G.S. topographic quadrangle West Granville. The dam is located at latitude N 42°-07'-34", longitude W 72°-53'-37". See Locus Plans 1 and 2.

(b) Description of Dam and Appurtenances

The dam consists of a hydraulic fill earth embankment about 730 feet long and 250 feet high. A free overflow concrete spillway weir and outflow channel is located about 1500 feet to the south, beyond Cobble Mountain. An intake tunnel control structure is located south of the spillway about 3000 feet south of the dam. A diversion tunnel intake control structure is located about 1000 feet northwest of the dam.

- 1) Embankment (B-1, B-2, B-3, B-4, B-5, B-6, B-7, B-8, B-9, and B-10)

The dam embankment, which has no appurtenant structures within its mass, is a sluiced hydraulic fill with a 1510 feet base width along the stream and 50 feet wide at the top with a top length of about 730 feet. A rock diversion dam with an impervious zone seal at the upstream face forms the upstream toe of the dam. A rock foundation drain fills the bottom of the valley to a depth of about 30 feet from a point about 200 feet downstream of the cutoff walls to the concrete arch toe dam at the downstream toe of the dam. The concrete toe dam is provided with three 4 ft. x 6 ft. weep holes to allow exit of foundation drainage. Topsoil was stripped up to 125 feet wide and a concrete cutoff wall was placed to a depth of at least five (5) feet into sound ledge rock across the valley and up to an elevation of 955. A second cutoff wall was placed across the valley bottom up to an elevation of about 765. In addition, extensive foundation grouting was carried out along the cutoff walls. Sluiced hydraulic fill was placed up to about elevation 945 and a puddled core up to elevation 956. The upstream and downstream embankment slopes were designed and constructed with variable slopes which begin fairly flat at the toe and increase in steepness toward the top of the dam. The embankment slopes vary from 1 on 4.72 and 1 on 4.52 at low levels to 1 on 1.5 above about elevation 959. The design drawings (Appendix B) indicate the design top of dam elevation was 965, however, during construction this was increased and a roadway was constructed across the top of the dam. Dam embankment construction was carried up to elevation 969.5 and road construction completed the embankment to elevation 973. Measurements made during this inspection indicate that the lowest point along the top of the dam is presently about elevation 972±.

Substantial rock fills were provided at both upstream and downstream toes of the embankment. The upstream face was covered with five (5) feet of rock to elevation 910 and 15 feet of rock above that elevation. The downstream face was provided with a five (5) foot thick cover up to elevation 860. Then the rock cover thinned to two feet thick from elevation 910 to elevation 956. Above elevation 956 the rock face became thicker as the sides were steepened to raise the top from design elevation of 965 to final top of road at elevation 973.

2) Spillway (B-1, B-11, B-12, B-13, and B-14)

The spillway is an open rock cut up to fifty (50) feet deep and fifty (50) feet wide at the bottom. A reinforced concrete overflow weir 135 feet long controls overflow. Weir crest is at elevation 952.0 feet NGVD. Concrete abutments rise six feet above the weir crest which stands six feet above the sand floor approach channel and concrete lined spillway chute. The concrete lined spillway chute narrows to about fifty (50) feet wide at the roadway bridge which is 175 feet downstream from the weir and extends past the bridge to a point about 210 feet downstream from of the weir.

The spillway bridge is an elliptical concrete arch thirty (30.00) feet wide with a sixty (60) foot span at spring line and an elevation of 967 NGVD at the soffit of the crown.

(3) Intake (B-16, B-17, B-18, and B-19)

The intake control tower is a 32 ft. x 28 ft. reinforced concrete structure housing 20 ft. x 20 ft. trash rack and roller gate and hoisting machinery. The structure also contains a six (6) foot diameter control valve and its emergency hydraulic operating pump, as well as a 7 ft. x 13 ft. air vent and tunnel access shaft. Principal operating power is electric, remotely controlled at the powerhouse and filter plant. Emergency power for the roller gate hoist is a battery at an upper level of the control tower. Emergency operation of the 6 ft. diameter valve is by a hand operated hydraulic pump.

A 10.0 ft. wide x 11.5 ft. high concrete lined tunnel with an invert at elevation 830 and 218 feet long connects the control tower with the reservoir. A concrete lined tunnel 10.0 ft. wide x 9.33 ft. high and 6350 ft. long connects the control tower to a 10 ft. diameter steel power plant inlet that delivers water to the hydroelectric power house at the Intake Reservoir.

(4) Diversion (B-15)

A concrete lined 10.5' x 10.5' horseshoe section tunnel 1600 ft. long allows stream diversion and reservoir draining

around the dam embankment. The tunnel floor at the reservoir inlet is at elevation 754. Control is via two 42 in. diameter steel pipes and valves at elevation 753.25 ft. MSL at centerline. Each pipe is fitted with a 39-3/4 inch rotary valve by Escher-Wyss with 4 in. bypass and then a 42 in. x 30 in. Larner-Johnson discharge regulator discharging through a brick wall into the discharge tunnel which has an air vent and separate stairway from the valve room to the surface. The valves are operated by hand wheels from an operating floor above the valve room.

(c) Size Classification

Both the structural height of 250 feet and capacity of 96,500 acre feet put this dam in the LARGE size classification.

(d) Hazard Classification

The extensive flooding and great loss of life in Westfield and West Springfield that would result from failure of this dam places it in the HIGH hazard classification. There is a high potential for severely damaging hundreds of homes with the attendant possible loss of hundreds of lives as well as numerous highway and railroad bridges, industrial and commerical areas, and other structures.

(e) Ownership

Cobble Mountain Reservoir Dam is owned by the City of Springfield, represented by its Board of Water Commissioners. The address is as follows:

City of Springfield Water Department  
City Hall, Court Square  
Springfield, Massachusetts 01105  
Tel: 413-787-6060

(f) Operator

Cobble Mountain Reservoir Dam is operated by the City of Springfield, acting through its Water Department.

The operator at the dam site on a daily basis is:

Mr. William York  
Borden Brook Reservoir  
Granville, Massachusetts 01034  
Tel: 413-357-8733  
Emergency Tel: 413-787-6206

(g) Purpose of Dam

The purpose of Cobble Mountain Reservoir Dam is to provide a water supply reservoir for the City of Springfield, Massachusetts. The reservoir also provides for hydroelectric generation.

(h) Design and Construction History

Cobble Mountain Reservoir Dam was designed by Mr. Allen Hazen of New York and built during the period 1927 to 1932 under multiple construction contracts.

Notes of Mr. James L. Tighe who reviewed the design and construction for the Hampden County Commissioners indicate that close control was maintained over the quality of the material being sluiced into the hydraulic fill core. On May 20, 1930 sluicing operations were stopped because of excessively fine material. Construction methods and borrow areas were reorganized and construction resumed on September 20, 1930. Construction proceeded steadily thereafter with adjustments of borrow material from time to time and, on occasion, removal of unsatisfactory material from the core. Core construction was completed October 17, 1931 and rock paving was completed on January 30, 1932 when the reservoir water level was at elevation 832. Diversion tunnel gates were closed on August 25, 1931 to start reservoir filling. The City of Springfield started using water from Cobble Mountain Reservoir on December 1, 1931. Road construction was completed at a later date.

A seepage weir with a crest length of 4.0 feet was constructed downstream of the dam. James L. Tighe's notes indicate that seepage increased as the reservoir filled. The reservoir reached an elevation of 938.7 on January 31, 1933 when flow over the seepage weir was 0.98 cfs. Seepage flow during 1933 and 1934 correlated closely with reservoir levels. On June 16, 1934 with the reservoir at elevation 939.6, the seepage flow was 0.86 cfs.

The dam was designed to have a top elevation of 965.0 feet MSL, a top width of 50 feet, and variable embankment slopes with a maximum slope downstream of 1 on 2 $\frac{1}{4}$  and 1 on 2-3/4 upstream. The top of the dam was raised during construction to a final elevation of 969.5 before road construction by increasing the steepness of the upper part of the dam to 1 on 1 $\frac{1}{2}$  both upstream and downstream.

The spillway was designed to have a crest in the concrete floor at elevation 945.0 with flashboards mounted on steel pipes seven feet high to elevation 952.0. The concrete spillway weir was actually constructed to an elevation of 952.0 with no provision for flashboards. At the same time, the concrete spillway abutments were raised from elevation 955.0 to elevation 958.0.

Maintenance grouting has been carried out in the diversion tunnel to control seepage. In 1973, the spillway weir wall was reconstructed and the concrete outflow channel floor and side walls were resurfaced and weepholes were installed.

(i) Normal Operating Procedures

The dam, intake works, supply tunnel, diversion tunnel and diversion control structure are owned and maintained by the City of Springfield. Daily operation of the power plant which regulates reservoir level is under control of Northeast Utilities. The operating agreement specifies minimum reservoir levels for each month of the year based on annual water demand by the City of Springfield. Minimum daily discharge required to meet the requirements of the City of Springfield is released. Within these limits the power company operates to use all inflow to the reservoir. Normally there will be no flow over the spillway. Normally, the reservoir will be within about a foot of the spillway crest in late spring and at its lowest level in late fall. There are no special operations for flood control purposes.

1.3 Pertinent Data

(a) Drainage Area

The drainage area tributary to this dam is about 45.8 square miles including 8 square miles tributary to Borden Brook Reservoir which is upstream of Cobble Mountain Reservoir. Most of the area is steep and mountainous with good forest cover.

Controlled flow from the watershed of the Littleville Flood Control Dam can be diverted to the Reservoir via a pumping station located in Russell.

(b) Discharge at Dam Site

1) Outlet Works

Normal discharge from the reservoir is via a ten (10) foot diameter concrete lined tunnel which delivers water to a hydroelectric powerhouse. The invert of the tunnel intake is at elevation 830.0 MSL. Inlet controls consist of a 6 foot diameter valve and a 20 ft. x 20 ft. steel roller gate which are installed in series with the steel roller gate on the upstream side of the intake structure. The intake tunnel has a capacity of approximately 1120 CFS with the reservoir level at the spillway test flood elevation of 967.3.

A reservoir drain and diversion tunnel consisting of a 10½ ft. x 10½ ft. concrete lined tunnel has inlet invert at elevation 754.0 feet MSL. Control is via two 42 in. diameter steel pipes. Each pipe discharges through two valves in series into the outlet tunnel. The diversion tunnel has a capacity of approximately 3780 CFS with the reservoir level at the spillway test flood elevation of 967.3.

2) Maximum Known Flood

The maximum flood record at this site occurred on August 19, 1955. Peak inflow has been estimated to be about 19,600 cfs. Peak outflow via spillway and intake tunnel has been estimated at 12,850 cfs. The reservoir stage reached a maximum elevation of 959.1 feet NGVD, which is 7.1 feet above the spillway crest.

3) Ungated Spillway Capacity at Top of Dam

The capacity of the spillway above the crest elevation (952.0 feet NGVD) to the top of the dam (elevation 972. $\pm$  feet NGVD) is about 55,000 cfs.

Elevation 972 $\pm$  is the low point along the roadway over the top of the dam as field measured during this inspection.

4) Ungated Spillway Capacity at Test Flood

The capacity of the spillway above the crest elevation (952.0 feet NGVD) with the reservoir at test flood elevation (967.3 feet NGVD) is about 38,000 cfs.

5) Gated Spillway Capacity at Normal Pool

None

6) Gated Spillway Capacity at Test Flood

None

7) Total Spillway Capacity at Test Flood

The capacity of the spillway above the crest elevation (952.0 feet NGVD) with the reservoir at test flood elevation (967.3 feet NGVD) is about 38,000 cfs.

8) Total Project Discharge at Top of Dam

Total project discharge including spillway, intake tunnel and diversion tunnel with reservoir at top of dam elevation (972 feet NGVD) is about 60,000 cfs.

9) Total Project Discharge at Test Flood

Total project discharge including spillway, intake tunnel and diversion tunnel with reservoir at test flood elevation (967.3 feet NGVD) is about 42,900 cfs.

(c) Elevation (feet above MSL NGVD)

1) Stream bed at toe of dam: 722 $\pm$

2)	Bottom of cutoff:	710±
3)	Maximum tailwater:	Unknown
4)	Normal pool:	952.0
5)	Full flood control pool:	not applicable
6)	Spillway crest:	952.0
7)	Design surcharge: Based on:	958.5
	a) Design spillway crest	945.0
	b) Design top of dam	965.0
8)	Top of dam	
	a) as built	973
	b) existing low point	972±
9)	Test flood surcharge:	967.3
(d)	<u>Reservoir</u> (length in feet)	
1)	Normal pool (952.0 NGVD):	21,000
2)	Flood control pool:	Not applicable
3)	Spillway crest pool (952.0 NGVD):	21,000
4)	Top of dam (972 NGVD):	22,600
5)	Test flood surcharge (967.3 NGVD):	22,300
(e)	<u>Storage</u> (acre-feet)	
1)	Normal pool (952.0 NGVD):	70,000
2)	Flood control pool:	Not applicable
3)	Spillway crest pool (952.0 NGVD):	70,000
4)	Top of dam (972 NGVD):	96,500
5)	Test flood pool (967.3 NGVD):	91,200
(f)	<u>Reservoir Surface</u> (acres)	
1)	Normal pool (952.0 NGVD):	1134
2)	Flood control pool:	Not applicable

- 3) Spillway crest pool (952.0 NGVD): 1134
- 4) Top of dam (972.1 NGVD): 1445
- 5) Test flood pool (967.3 NGVD): 1385

(g) Dam

- 1) Type: Hydraulic fill between rock toes
- 2) Length: 730 feet
- 3) Height: 250 feet
- 4) Top width: 50 feet
- 5) Side slopes: upstream: 1:4.72 to 1:1.5  
downstream: 1:4.52 to 1:1.5
- 6) Zoning: sluiced hydraulic core  
rock fill toe upstream and downstream  
rock facing upstream and downstream  
rock riprap upstream upper zone
- 7) Impervious core: sluiced hydraulic fill
- 8) Cutoff: 3 ft. wide x 3 ft. high concrete at least 5 ft.  
deep in good rock. One wall full width of  
dam, one wall across valley bottom
- 9) Grout curtain: 15 ft. deep full valley width and 8 holes  
25 feet deep

(h) Diversion and Intake Tunnels

	<u>Diversion</u>	<u>Intake</u>
1) Type:	concrete lined rock bore	concrete lined rock bore
2) Length:	1550 feet	7500 feet
3) Closure:	Concrete 48 feet long	Inlet control structure
4) Access:	Gatehouse shaft	Gatehouse shaft
5) Regulating Facilities:	2-42" steel pipes each with 1 rotary valve and 1 retracting plug valve	20' x 20' steel roller gate 6' diam. valve & turbine controls @ power plant

i) Spillway

- 1) Type: Concrete wall 6 ft. high, 4 in. x 4 in. chamfered front corner, rounded downstream corner of 3 ft. wide top.
- 2) Length of weir: 135.0 feet
- 3) Crest elevation: 952.0 NGVD
- 4) Gates: none
- 5) Upstream channel: sand and gravel
- 6) Downstream channel: concrete lined for 210 ft. then rock cut
- 7) General: Channel floor drops 15 ft. in 200 ft. and narrows from 135 ft. to 50 ft., then 1% slope in rock cut.

(j) Regulating Outlets

		<u>Diversion</u>	<u>Intake</u>
1)	Invert	754.0	830.0
2)	Size	2-42 in. in parallel	20 ft. x 20 ft. 6 ft. diam.
3)	Description	40 in. rotary retracting plug	roller gate valve
4)	Control mechanism	handwheels	electric hoist hydraulic operator

## SECTION 2 - ENGINEERING DATA

### 2.1 Design Data

Design data shown on eighteen sheets of design plans and reports of the County Commissioner's Engineer and Review Board indicate the dam was designed in accordance with good engineering practice of the time.

The site was well mapped and investigated for foundation and embankment material. Project facilities were located to assure maximum integrity of the dam embankment. There were no abrupt changes in the regular "V" shaped valley profile that would indicate potential problems. Drainage was specifically provided for. Generous spillway capacity for that time - 20,000 cfs - was provided at a site removed from the embankment. Generous, secure diversion and intake works were provided.

### 2.2 Construction Data

The diary and notes of James L. Tighe who inspected the work for the Hampden County Commissioners, other review commission reports, and published reports indicate that construction was carefully supervised to assure that good quality material was properly placed during construction.

### 2.3 Operation Data

The dam, intake works, supply tunnel, diversion tunnel and diversion control structures are owned and maintained by the City of Springfield. Daily operation of the power plant which regulates reservoir level is under the control of Northeast Utilities. The operating agreement specifies minimum reservoir levels for each month of the year based on annual water demand by the City of Springfield. Minimum daily discharge required to meet the requirements of the City of Springfield is released. Within these limits, the power company operates to use all inflow to the reservoir. Normally there will be no flow over the spillway. Normally the reservoir will be within about a foot of the spillway crest in late spring and lowest in late fall. There are no special operations for flood control purpose.

Maintenance records are available from the City of Springfield. Detailed records of operation including reservoir elevation, discharge, etc., are available from Northeast Utilities, the power plant operator.

### 2.4 Evaluation of Data

#### (a) Availability

Plans of the dam as built and basic information were readily available from the Springfield Water Department. Additional reports were readily available in publications of the New England Water Works Association. Notes and design plans and reports were

obtained from the files of Tighe & Bond/SCI, Easthampton, Massachusetts. Records of materials placed during construction are available from the Springfield Water Department.

(b) Adequacy

The data and information was adequate, when combined with visual inspection and engineering experience and judgment, to evaluate the safety of Cobble Mountain Reservoir Dam under normal operating conditions. The data examined for this report was not adequate to evaluate the safety of Cobble Mountain Reservoir Dam under seismic load conditions. There may not be adequate existing information for seismic analyses, including the potential for liquification of the hydraulic fill.

(c) Validity

Since visual inspection observations confirm the available data and reports from several independent sources concur on construction procedures, it is concluded that the data is valid and reliable.

## SECTION 3 - VISUAL INSPECTION

### 3.1 Findings

#### (a) General

Cobble Mountain Reservoir Dam is in GOOD condition at the present time.

#### (b) Dam (see photos in Appendix C)

The dam embankment was found to be covered with rock. The upstream face had a thick layer of rock that obscured any observation of underlying material. The downstream face was not as thickly covered and gravel and sand were at the surface in some areas. Brush growth on all faces of the dam had been cut and was obviously well controlled. Drainage through the concrete toe dam was clear and free from silt though red iron oxide sludge was present in the bottom of pools. Rock debris filled the bottom of the valley nearly to the top of the weep holes. There was no sign of leakage from the face of the dam above the toe dam. The concrete toe dam showed weather deterioration up to 2 inches deep on all exposed faces. There was minor seepage from the face of the toe dam near the right abutment. The road across the top of the dam was in good condition. The road at the center of the dam was observed to be about 1.8 feet lower than the approaches to the dam.

This difference is believed to be due to a combination of minor settlement of the dam embankment and the formation of a vertical curvature in the roadway with the abutment areas being built up forming the roadway approaches. The abutment areas are believed to be somewhat higher than elevation 973 due to the roadway construction.

#### (c) Appurtenance Structures

The spillway was found to be in good condition. The spillway weir wall was found sound and in good condition with only minor hairline cracks. Concrete sidewalls and floor had been repaired. Minor seepage was apparent at some joints. Weep holes appeared to be functioning. The spillway bridge was found in excellent condition. The lower part of the spillway with natural rock walls was found with considerable growth of brush up to 2 inches in diameter that would have no detrimental effect on the operation of the spillway.

The intake control building was found in excellent condition. Submerged portions of the structure and equipment were not inspected. Equipment appeared to be in good condition except for a leaky hand hydraulic pump and a jerry-rigged float well for depth indication. Emergency operating facilities appeared to be adequate. The diversion tunnel access building was found in good

condition. Windows were boarded for security against vandalism. Only minor leakage and efflorescence was observed in lower portions of the access shaft. The valve operating floor was in good condition with operating handwheels for all valves in position and no significant deterioration. The valve chamber was found in good condition with no significant deterioration. All valves appeared to be operable; no operation was tested. The valves are frequently operated as part of the powerhouse operation and for routine inspection and maintenance. The diversion tunnel was found in good condition. Some debris from trespass intruders was observed, but none that would prevent normal function. Some leakage and efflorescence was observed in the 340 feet of tunnel nearest the outlet end. Grout holes and evidence of grouting work was noted. The outlet portal showed weather deterioration up to 2 inches deep. A steel grate barrier had been installed but was not maintained closed.

(d) Reservoir Area

The reservoir and shore line appeared to be reasonably clear up to spillway elevation with only scattered brush and minor debris noted at any location.

(e) Downstream Channel

In the valley bottom downstream of the dam to the powerhouse the only flow is leakage through the dam and rare spillway overflows below the spillway. The valley floor was found to have a substantial growth of brush and small trees up to 3 inches in diameter.

### 3.2 Evaluation

The dam and appurtenances are generally in excellent condition. The maintenance staff is to be complimented.

The following deficiencies or problems may warrant attention:

1. A proper float well and depth indicator should be installed.
2. The hand operated hydraulic pump should be replaced or repaired and located above possible high water.
3. The diversion tunnel outlet gate should be removed if it is not to be maintained secure to avoid the possibility that debris inside the tunnel may partially block the outlet and cause back pressure at the valve chamber.
4. The present elevation and profile of the top of the dam should be determined.

## SECTION 4 - OPERATION AND MAINTENANCE PROCEDURES

### 4.1 Operation Procedures

See Section 1.2(i), page 1-5, for a general description of operating procedures.

There are no special operations for flood control purposes or other dam failure contingencies. The Springfield Water Department has an "Emergency Operations Plan" for dealing with general emergencies concerning the water supply and distribution system.

### 4.2 Maintenance Procedures

Regular maintenance is performed by a full time staff under the supervision of a foreman who lives at the reservoir. Daily visits are made to the intake control building and dam site. Regular maintenance includes:

Cleaning, lubrication and maintenance of intake control equipment;

Cleaning and maintenance of intake control building and diversion tunnel control building;

Clearing brush from dam, spillway and reservoir area;

General maintenance and debris removal from the reservoir, public areas, and City-owned watershed areas; and

An annual shutdown inspection and maintenance of power facilities and intake tunnel.

No formal program of technical inspections by the Springfield Water Department of the dam, spillway, and diversion tunnel is in effect.

### 4.3 Evaluation

The framework of normal operating procedures is adequate except for the following deficiencies:

1. A downstream emergency flood warning system should be developed and put into operation.
2. Generation and release operations responding to storm or flood warnings should be developed and put into practice.
3. A biennial program of technical inspections by a registered professional engineer qualified in dam design and inspection should be developed and put into practice.

## SECTION 5 - EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES

### 5.1 General

Cobble Mountain Reservoir Dam is located in Russell, Massachusetts on the Little River about 13 miles above its confluence with the Westfield River in Westfield, Massachusetts. The reservoir is about eight miles west of Westfield at the eastern margin of the Berkshire Hills. The drainage area of about 45.8 square miles is in mountainous terrain that varies to areas of rolling terrain in upper elevations. Bedrock is mica shist covered with glacial deposits of great variety and generally of low permeability. About 80% of the watershed is covered with good forest.

Downstream of the dam, the Little River valley has a steep gradient of about 2% and a steep "V" shaped ravine of about 1:2 to 1:1.2 slope. This valley extends downstream past the power house and intake dam about eight (8) miles to the eastern escarpment of the Berkshire upland at the rolling valley floor of the western part of Westfield. From this point, the river flows through a flood plain by the south margin of the City about five (5) miles to the Westfield River at the east end of the City of Westfield. The Westfield River winds about 3 miles across a flood plain until it enters the water gap between East Mountain and Provin Mountain at the West Springfield town line. For the next 5½ miles the Westfield River flows through a narrow channel between West Springfield and Agawam to the west edge of the Connecticut River flood plain. The Westfield River then flows more than a mile across the Connecticut River flood plain to empty into the Connecticut River less than a mile upstream from the Springfield-Longmeadow town line.

The spillway of the dam is a reinforced concrete wall standing six (6) feet above the approach and discharge channel floor. The spillway weir is 135 feet long. The discharge chute converges to 50 feet wide and drops in elevation about 15 feet in 200 feet and flows through a deep rock cut 50 feet wide to discharge into the Little River valley about half a mile below the toe of the dam.

The flow into the Cobble Mountain Reservoir can be augmented by water pumped from the Littleville Dam impoundment via a pumping station and pipeline to Cobble Mountain. The pumping station is located within the Town of Huntington and presently is equipped with 3 pumps having a combined capacity of about 45 MGD (million gallons per day). Provisions have been made for the future installation of a fourth pump which would increase the capacity by an additional 15 MGD.

### 5.2 Design Data

Design plans provide a spillway discharge profile and hydraulic calculations for a flow of 20,000 cfs, reported to be 3½ times record flow prior to design date. Page B-11 of Appendix B indicates a design water elevation of 958.5 feet at a 20,000 CFS discharge rate. Reports indicate that model studies were made of the proposed spillway and modifications indicated thereby were adopted. The design plans provide for a concrete crest threshold flush with the spillway floor in an

unlined rock channel. The spillway was actually constructed with concrete lining for about 200 feet from the crest through the bell mouth construction past the bridge. The six foot high concrete weir wall was constructed at a later date. A detailed technical analysis of downstream effects due to dam failure was not carried out in conjunction with the design of this dam based on records available for review.

### 5.3 Experience Data

Reports made at the time of design indicate flows of about 5700 cfs had been reported at this site and the storm of December 1878 was reported to yield runoff of about 150 cfs per square mile on the Westfield River.

A report by the Springfield Water Department on the storm of August 1955 indicates that the maximum inflow to the reservoir was about 19,600 cfs and the maximum discharge was about 12,850 cfs. The reservoir surface was 7.1 feet above the spillway crest wall. (Reservoir Stage = 959.1 NGVD)

### 5.4 Test Flood Analysis

The objective of the test flood analysis is to assess the capacity of the dam to safely pass a severe runoff event of a size commensurate with the size of the dam and the downstream hazard to life and property.

Guidelines for establishing a test flood are specified in "Recommended Guidelines" of the Corps of Engineers. The height of this dam (250 feet) and storage volume (96,500 acre-feet) put this dam in the LARGE size classification. The potential for flooding most of the City of Westfield below the elevation of Noble Hospital puts this dam in the HIGH hazard class. Table 3 of the Corps of Engineers "Recommended Guidelines" indicated that the spillway test flood for a LARGE size, HIGH hazard dam should be the probable maximum flood. The spillway test flood applied was PROBABLE MAXIMUM FLOOD.

Storm runoff of a maximum probable flood was estimated based on U.S. Corps of Engineers, New England Division "Preliminary Guidance for Estimating Maximum Probable Discharges." A discharge of 1440 cfs per square mile on the drainage area of 45.8 square miles gives a peak inflow of 66,000 cfs. Routing the probable maximum flood through the reservoir and spillway by preliminary methods assuming the initial reservoir level at spillway crest elevation (952.0 ft. NGVD), results in a maximum reservoir stage of about elevation 967.3 feet NGVD or about 4.7 feet below the top of the dam. Maximum discharge over the spillway would be about 38,000 cfs.

Analysis was made to evaluate the possibility of overtopping Cobble Mt. Dam if Borden Brook Reservoir Dam failed. If Borden Brook Dam failed during a storm with Cobble Mt. Reservoir at elevation 960 (8 feet above the spillway and higher than any past reservoir elevation) the maximum stage of Cobble Mt. Reservoir would be elevation 966.7 ft. NGVD which is 14.7 ft. above spillway crest, and 5.3 ft. below top of dam. Maximum discharge would be about 35,800 cfs over the spillway.

It is concluded that the spillway capacity is adequate to prevent overtopping of the dam due to a Probable Maximum Flood event.

### 5.5 Dam Failure Analysis

The hazards and potential damages resulting from failure of Cobble Mountain Reservoir Dam were evaluated assuming reservoir level at test flood elevation (967.3 feet NGVD) by the procedures suggested in New England Division, Corps of Engineers "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs." No allowance was made for possible clogging of waterways caused by trees and debris. The length of the dam at mid height and the height of the dam were taken from the plans of the dam provided by the Springfield Water Department. The distance across the valley was taken as 370 feet at elevation 850 and the dam height as 250 feet. The peak discharge using the suggested "Rule of Thumb" is 904,000 cfs. This flow was routed downstream to the Connecticut River.

The PMF discharge via the spillway of 38,000 cfs will flow down the Little River canyon at a depth of about 19 feet and velocities of about 24 fps. This flow would overtop the Intake Dam non-overflow sections by about six (6) feet probably causing considerable damage though not complete collapse. There would probably be some damage to the powerhouse. Dam failure flow could be as much as 90 feet deep in the valley and would probably destroy the powerhouse and possibly the Intake Dam at hazard area (2).

At Northwest Road - hazard area (3) - the PMF spillway discharge would probably flood the road across the valley about eight (8) feet deep and flood two (2) houses and about 250 acres of land. Dam failure flow would inundate Northwest Road as deep as 31 feet, flooding about six (6) houses and 400 acres.

At Horton's Bridge, where Granville Road crosses Little River - hazard area 4 - the PMF spillway discharge would probably flow over the road three (3) feet deep flooding the factory and about five houses above the dam there. Cobble Mountain Dam failure would result in water over the road to a depth of about 41 feet and the flooding of the mill and about 110 houses.

At the Route 202 bridge over the Little River - hazard area 5 - the PMF spillway flow of about 38,000 cfs from the Little River could result in flood water reaching the top of the road with little damage. Cobble Mountain Dam failure would result in flood waters up to about 12 feet above the road, flooding the mill and about 125 houses.

At hazard area 6, the former New Haven railroad embankment and bridge constitutes such a barrier and restriction that the PMF spillway flow would probably flow through the center of Westfield at depths of as much as five feet above some of the City streets. About 400 acres of the City might be flooded, the water reaching an elevation of about 149 feet MSL. The failure of Cobble Mountain Dam would create a flood wave about 19 feet higher than the PMF flood. This failure would flood as much as 600 acres, reaching elevations of about 168 feet MSL, nearly the elevation of Noble Hospital (Elev. 170).

At hazard area 7, the PMF spillway flow would rise as high as about elevation 125 feet MSL before it could pass through the water gap at the West Springfield - Westfield town line. This would inundate about 800 acres in Westfield and Southwick. Cobble Mountain Dam failure would result in water rising to elevation 157 feet MSL, inundating up to 3600 acres in a wide area.

At Dewey Street and Westfield Road - hazard area 8 - the PMF spillway flow would possibly rise to three feet above the road and flood 12 houses and buildings in the area. Dam failure could result in water up to eighteen feet deep over Route 20, Westfield Road, flooding about 30 houses and buildings.

At Front Street in West Springfield - damage area 9 - damage would probably be light as even the dam failure flood would be well below the railroad at Front Street and Bridge Street.

At the Memorial Drive bridge to Agawam - hazard area 10 - the PMF spillway discharge from the dam would pass satisfactorily but the dam failure peak flow would probably overtop the Westfield River flood protection dike by about two feet which could result in flooding about 210 acres of low land near the Eastern States Exposition.

Failure of Cobble Mt. Reservoir Dam would probably cause considerable flooding of low lying areas along the Connecticut River that are not protected by dikes - hazard area 10.

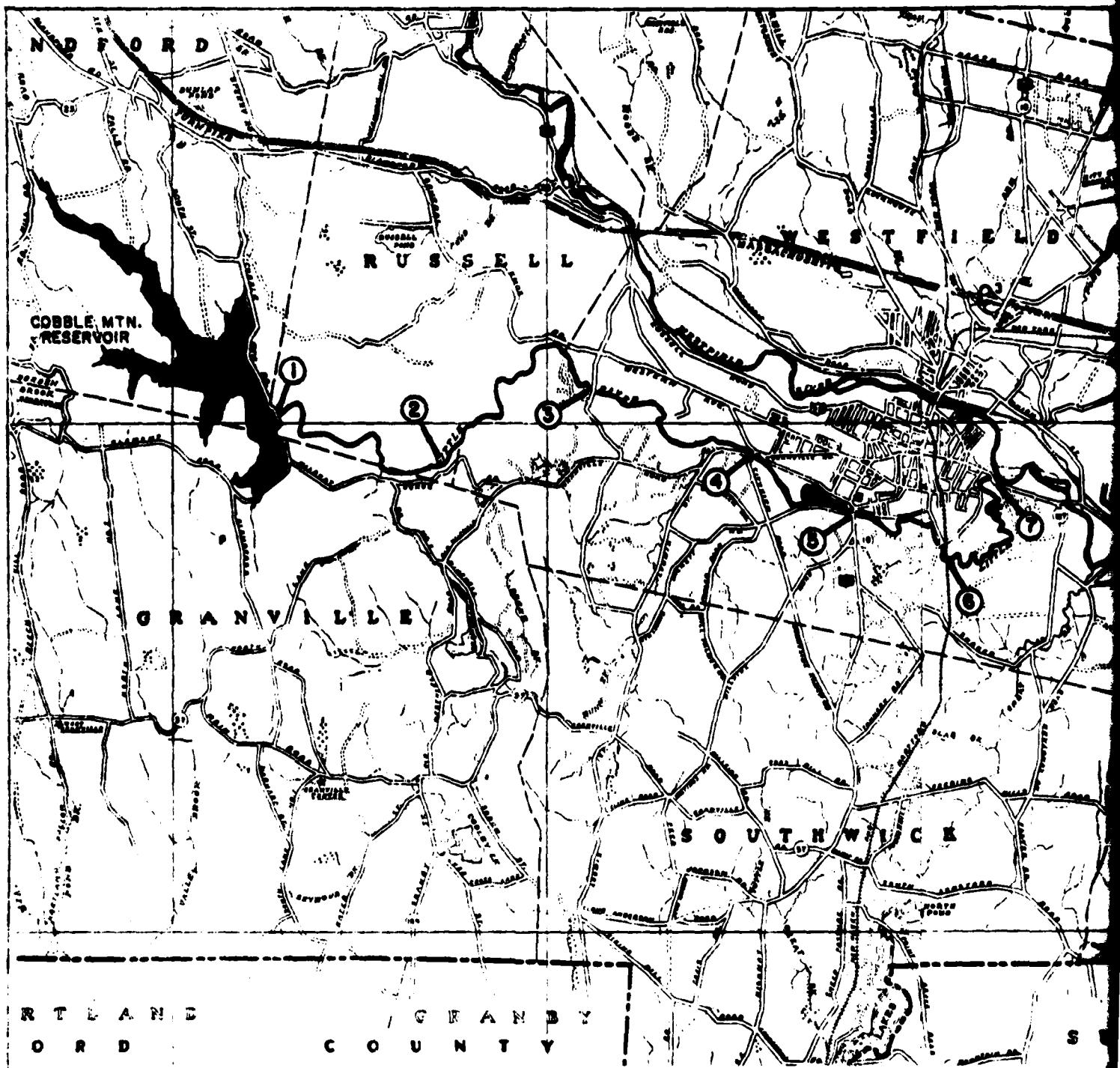
The information reviewed above is summarized in the following table of "Downstream Impacts of Dam Failure," the "Location and Hazard Map," and more fully developed in the computations in Appendix D.

In summary, a Cobble Mountain Dam failure would result in extensive flooding and great loss of life in Westfield and West Springfield. There is a high potential for severely damaging hundreds of homes with attendant possible loss of hundreds of lives as well as numerous highway and railroad bridges, industrial and commercial areas, and other structures.

DOWNTSTREAM IMPACTS OF DAM FAILURE

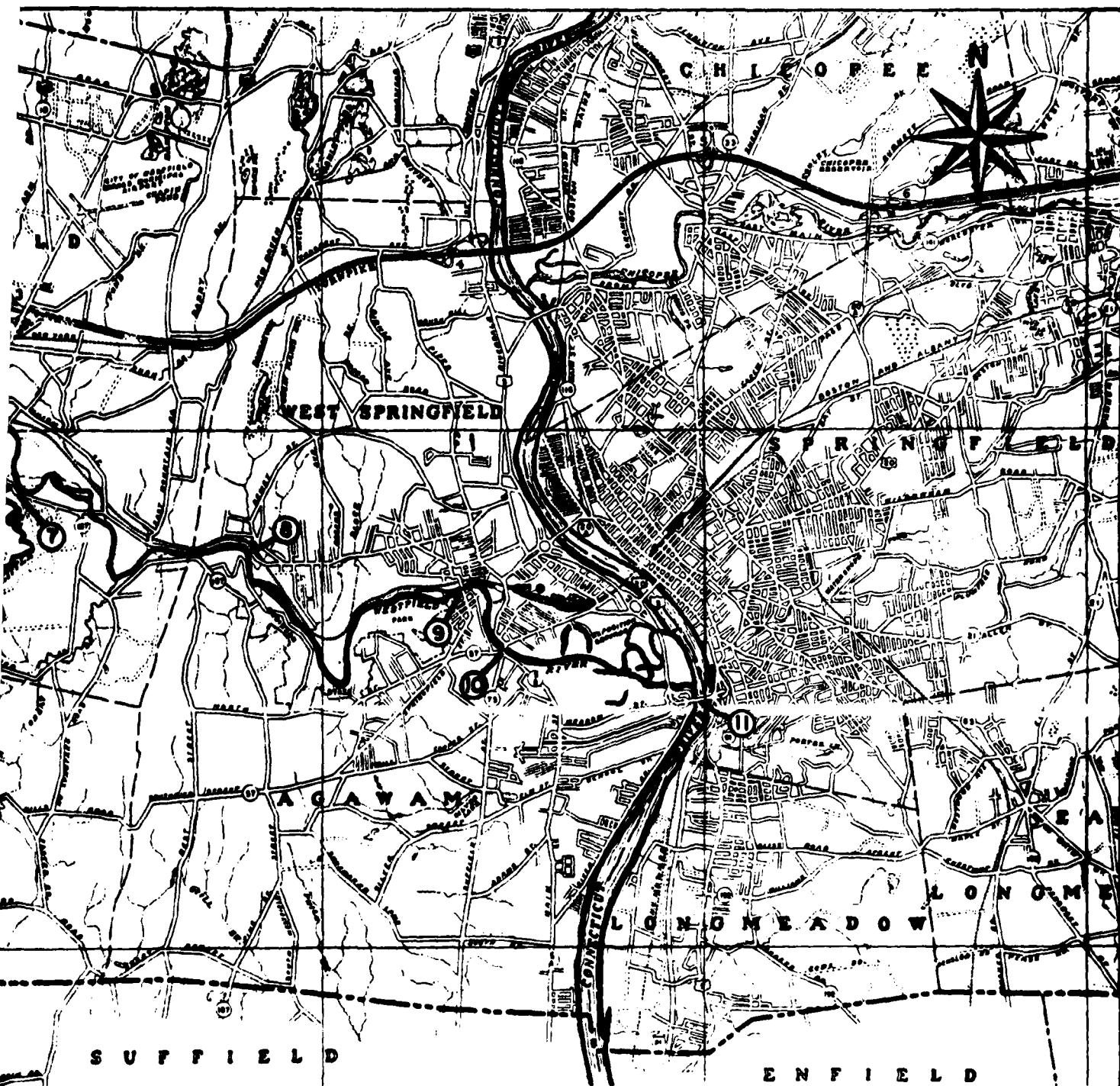
Map Location No.	Distance From Dam ft.	Feature	Before Dam Failure Flood Stage cfs	Depth Over Road ft., elev.	Bldgs-Area Flooded each, acres	After Dam Failure Flood Stage cfs	Depth Over Road ft., elev.	Bldgs-Area Flooded each, acres	Comments
1	0	Dam	38,000	---	---	904,000	---	---	---
2	14,000	Intake Dam	38,000	---	---	861,000	70	1	Power Station flooded by dam failure.
3	31,500	North-west Rd.	38,000	8	250 Ac.	802,000	31	6	
4	44,000	Horton's Bridge	38,000	3	5	720,000	41	400 Ac.	Mill at dam damaged by base flood.
5	52,000	Rte. 202 Bridge	38,000	---	---	677,500	12	125	Dam & mill destroyed by dam failure
6	59,600	N.H. R.R. Bridge	38,000	5	400 Ac.	605,000	24	257 Ac.	Main St., Elm St., Broad St., flooded by base flood to City Hall; by failure to Noble Hospital.
7	69,000	East Main St.	38,000	3	800 Ac.	210,000	35	3600 Ac.	Base flood over East Main St. Failure floods wide area.
8	88,700	Dewey St.	38,000	3	12	196,000	18	30	

Map Location No.	Distance From Dam ft.	Feature	Before Dam Failure			After Dam Failure			Comments
			Flood Stage cfs	Depth Over Road ft. - elev.	Bldgs-Area Flooded each acres	Flood Stage cfs	Depth Over Road ft. - elev.	Bldgs-Area Flooded each acres	
9	110,200	Front St.	38,000	---	---	194,000	26	---	No Damage
10	116,000	Agawam Bdg.	38,000	---	---	194,000	11 Elev. 72	210	Failure flood is 2' above flood dike.
11	129,000	So. End Bdg.	54,000	---	---	210,000	-- 15 Elev. 58	---	Dam failure floods unprotected low areas.



- ① : COBBLE MTN. RESERVOIR DAM
- ② : 14,600' D.S. INTAKE DAM
- ③ : 31,500' D.S. NORTHWEST ROAD
- ④ : 44,000' D.S. HORTON'S BRIDGE
- ⑤ : 52,000' D.S. ROUTE U.S. 202
- ⑥ : 59,600' D.S. RAILROAD BRIDGE

- ⑦ : 69,000' D.S. EAST MAIN ST., RTE. 20
- ⑧ : 88,700' D.S. DEWEY STREET
- ⑨ : 110,200' D.S. FRONT STREET
- ⑩ : 116,000' D.S. AGAWAM BRIDGE
- ⑪ : 129,000' D.S. SOUTH END BRIDGE



-SCALE-



FROM: GENERAL HIGHWAY MAP,  
HAMDEN COUNTY

TIGHE & BOND / SCI  
CONSULTING ENGINEERS  
EASTHAMPTON, MASS.

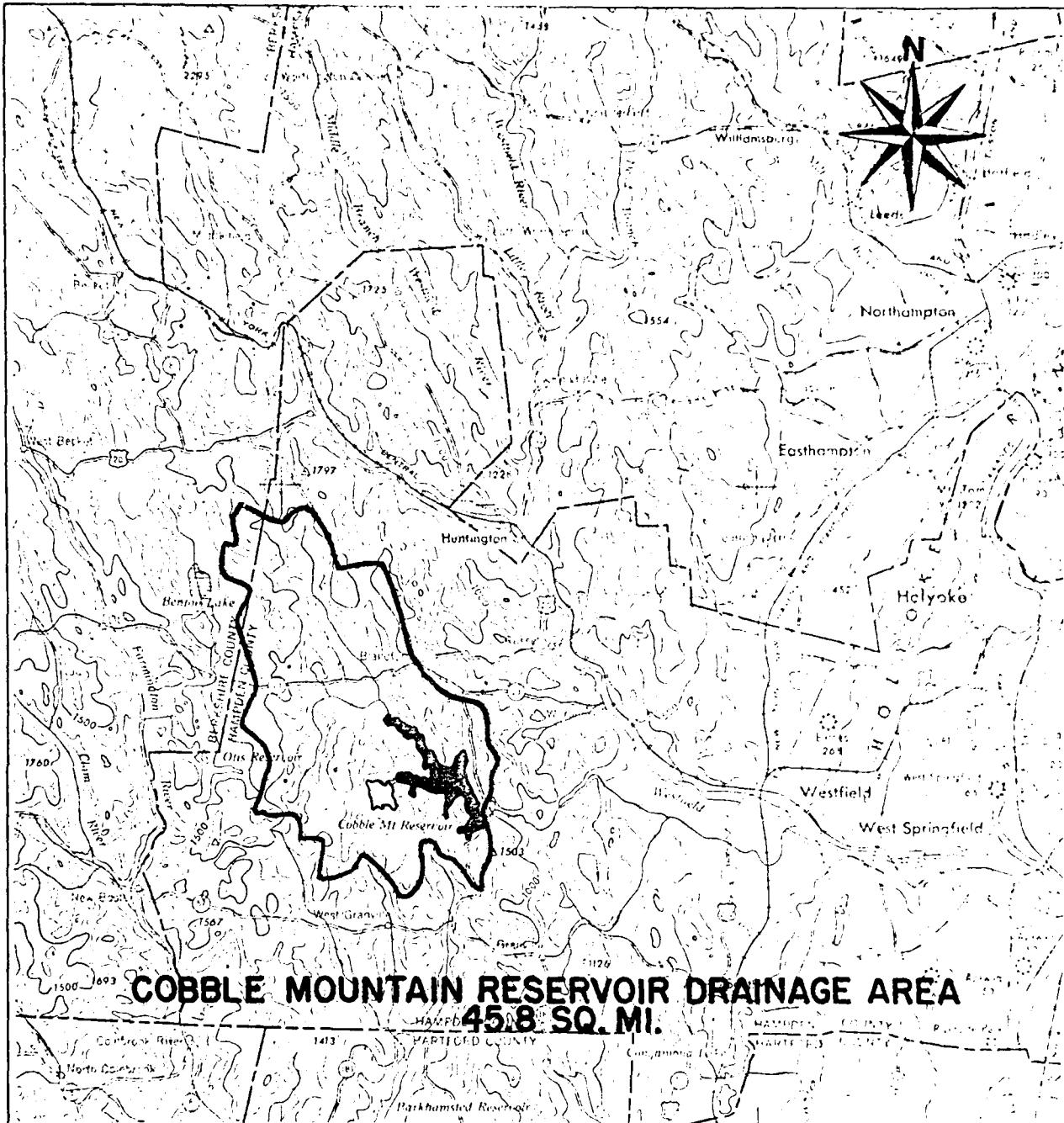
U.S. ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

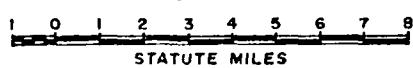
LOCATION AND DOWNSTREAM  
HAZARD MAP

COBBLE MOUNTAIN RESERVOIR DAM (MA 00068) RUSSELL  
HAMPDEN COUNTY MASSACHUSETTS

SCALE: AS NOTED  
DATE: MARCH 1980



-SCALE-



FROM: U.S.G.S. TOPOGRAPHIC MAP  
NK 18-6, ALBANY, UNITED  
STATES



TIGHE & BOND / SCI  
CONSULTING ENGINEERS  
EASTHAMPTON, MASS.

U.S.ARMY ENGINEER DIV. NEW ENGLAND  
CORPS OF ENGINEERS  
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

## DRAINAGE AREA MAP

COBBLE MOUNTAIN RESERVOIR DAM (MA 00068)      RUSSELL  
HAMPDEN COUNTY      MASSACHUSETTS

SCALE: AS NOTED

DATE: MARCH 1980

## SECTION 6 - EVALUATION OF STRUCTURE STABILITY

### 6.1 Visual Observation

Visual inspection of Cobble Mountain Dam did not reveal any signs of abnormal or alarming movement of the structure; nor signs of any significant leakage that should be cause for concern.

### 6.2 Design and Construction Data

Design and construction data and published reports provide information on the nature and arrangement of materials in the dam. The reports and information examined were not adequate to make a modern analysis of stability of the embankment or to evaluate seismic susceptibility of the dam. The plans and reports indicate that the best practice of the day was followed and no signs of distress have been observed.

### 6.3 Post Construction Changes

Near the end of construction, the slopes of the upper 14 to 17 feet were steepened from 1 on 2.25 and 1 on 2.75 to 1 on 1.5 to increase the height of the dam by seven (7) feet to elevation 973 feet MSL.

Sometime after construction was complete the flashboards were replaced with a reinforced concrete wall seven (7) feet higher than the design crest. This reduced the freeboard safety and ability to pass severe storms without overtopping the dam. It is estimated that even so, the project can pass a probable maximum flood safely.

Other post construction work, namely, diversion tunnel grouting and spillway concrete repair, have not altered the characteristics of the project, but have been carried out to maintain them in good condition.

The diversion tunnel outlet portal barrier grate which was installed to prevent trespass has not been successful.

The Massachusetts Division of Waterways' most recent inspection report dated May 2, 1978 was not available for review.

### 6.4 Seismic Stability

The dam is located in seismic zone No. 2. According to the recommended Corps of Engineers Guidelines, a seismic analysis would not normally be warranted. However, due to the susceptibility of hydraulic fill dams to failure due to earthquake motion and to the extremely high hazard to downstream communities in the event of dam failure, it is recommended that seismic stability studies including the potential for liquefaction of the hydraulic fill be undertaken.

## SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

### 7.1 Dam Assessment

#### (a) Condition

Cobble Mountain Reservoir Dam is generally in good condition. Leakage is not excessive nor is there any sign of piping or seepage that should be cause for concern. Maintenance is good and the project condition is excellent. No sign of structural distress was observed. However, due to the lack of a seismic stability analysis for this hydraulic fill dam, it has been rated as FAIR.

#### (b) Adequacy of Information

Information on soil and rock embankment materials in place that would be adequate to perform modern stability analyses or seismic analysis was not available. Information concerning design and construction was otherwise complete and adequate. Information regarding the current post construction elevation of the top of the dam was not available.

#### (c) Urgency

The recommendation for a seismic stability analysis should be initiated within one (1) year from receipt of this report by the Owner.

The remedial measures described herein should be implemented within two (2) years.

### 7.2 Recommendations

It is recommended that the following items be investigated under the supervision of a qualified registered professional engineer:

1. Determine the strength and stability of the embankment especially with respect to seismic loading conditions, including the potential for liqufaction of the hydraulic fill.

### 7.3 Remedial Measures

The following remedial, maintenance and operation procedures are recommended:

1. Develop an "Emergency Action Plan" that will include an effective pre-planned downstream warning system, locations of emergency equipment, materials and manpower, authorities to contact and potential areas that require evacuation.
2. Determine the current elevation and profile of the top of the dam.

3. Install a secure, reliable depth measurement float.
4. Correct leak and relocate hand hydraulic pump to an upper floor.
5. Remove diversion tunnel outlet security gate.
6. Institute a biennial technical inspection by a registered professional engineer qualified in dam design and inspection.

#### 7.4 Alternatives

There are no practical alternatives to the above stated Recommendations and Remedial Measures.

APPENDIX A  
INSPECTION CHECKLIST

## INSPECTION CHECK LIST

## PARTY ORGANIZATION

PROJECT Cobble Mountain Reservoir DamDATE 11/15/79TIME 8:30WEATHER Clear, breezyW.S. ELEV. 936 U.S. MSL D.N.S.PARTY: Tighe & Bond/SCIBased on design elevations of  
structures

- |   |                                 |
|---|---------------------------------|
| 1. <u>John W. Powers, Project Manager</u> | 6. <u>Omer H. Dumais, Civil</u> |
| Hydrology/                                |                                 |
| 2. <u>George H. McDonnell, Hydraulics</u> | 7. _____                        |
| 3. <u>Edward A. Moe, Soils/Hydraulics</u> | 8. _____                        |
| 4. <u>David M. Lenart, Civil</u>          | 9. _____                        |
| 5. <u>Howard A. Koski, Civil</u>          | 10. _____                       |

PROJECT FEATURE	INSPECTED BY	REMARKS
1. All project features were inspected by all party members.		
2.		
3.		
4.		
5.		
6.		
7.		
8.		
9.		
10.		

Present for the Owner: Francis Broderick, William York

## INSPECTION CHECK LIST

PROJECT Cobble Mountain Dam

DATE 11/15/79

PROJECT FEATURE Embankment

NAME Tighe &amp; Bond party

DISCIPLINE \_\_\_\_\_

NAME \_\_\_\_\_

AREA EVALUATED	CONDITIONS
<u>DAM EMBANKMENT</u>	
Crest Elevation	972.1 ± (35.8' ± above water level)
Current Pool Elevation	936.3 ± (15.7' ± below spillway crest)
Maximum Impoundment to Date	959.1 (7.1' above spillway crest)
Surface Cracks	None Concrete toe wall shows surface weathering & deterioration up to 2" deep
Pavement Condition	Road pavement - good
Movement or Settlement of Crest	0.9' based on recon. levels 1.8' based on road grade observations
Lateral Movement	None evident
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good, only minor erosion
Indications of Movement of Structural Items on Slopes	None. variations in downstream slope rock on gravel bed appears to be as built, not subsequent movement.
Trespassing on Slopes	Yes, some debris, no damage.
Vegitation on Slopes	Small brush. Maintenance clearing is good.
Sloughing or Erosion of Slopes or Abutments	No sloughing. Moss on rock & gravel indicate stability. Minor localized erosion along rock pocket at abutment.
Rock Slope Protection - Riprap Failures	Good condition both faces.
Unusual Movement or Cracking at or near Toes	None
Unusual Embankment or Downstream Seepage	No seepage at surface. Maximum passing flow estimated at about 1 cfs. Minor seepage through toe wall near rock face.
Piping or Boils	None. No silt in outflow.
Foundation Drainage Features	Rock foundation drain to toe wall & weep holes appear to be functioning well.
Toe Drains	3-6' wide weep holes - tops seen above rock. Outflow clear - no silt, some iron oxide sludge.
Instrumentation System	None

## INSPECTION CHECK LIST

PROJECT Cobble Mountain Dam

DATE 11/15/29

PROJECT FEATURE Diversion tunnel control tower

NAME Tighe &amp; Bond party

DISCIPLINE \_\_\_\_\_

NAME \_\_\_\_\_

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - CONTROL TOWER</u>	
a. Concrete and Structural	
General Condition	Excellent
Condition of Joints	Good
Spalling	None
Visible Reinforcing	None
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	Minor leakage & efflorescence below Level 8
Joint Alignment	Good. Brick wall around Larner-Johnson discharge nozzles
Unusual Seepage or Leaks in Gate Chamber	No. Only minor leakage
Cracks	None
Rusting or Corrosion of Steel	None
b. Mechanical and Electrical	
Air Vents	Good - 10'x4'-8" vent & hoist shaft
Float Wells	None
Crane Hoist	None fixed
Elevator	None
Hydraulic System	None
Service Gates	2-42" pipe & 42" x 30" Larner-Johnson discharge regulators
Emergency Gates	2-39-3/4" Escher-Wyss rotary valves & 4" bypass
Lightning Protection System	None
Emergency Power System	Valves are hand wheel operated from floor 10' above
Wiring and Lighting System in Gate Chamber	None except temporary drop light

## INSPECTION CHECK LIST

PROJECT Cobble Mountain DamDATE 11/15/79PROJECT FEATURE Diversion TunnelNAME Tighe & Bond partyDETAILEE NAME 

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL</u>	
General Condition of Concrete	Good at gate chamber to fair at outlet portal
Rust or Staining	None
Spalling	Minor, less than 1½" deep at some joints in last 430'
Erosion or Cavitation	None
Visible Reinforcing	None
Any Seepage or Efflorescence	Some last 430'. Gets worse toward outlet end. Localized near joints. Grout holes evident.
Condition at Joints	
Drain holes	None
Channel	
Loose Rock or Trees Overhanging Channel	No
Condition of Discharge Channel	Rocky & brushy with trees less than 3" Ø
Outlet Portal	Steel grate barrier & door, not locked. Concrete shows general surface deterioration

## INSPECTION CHECK LIST

PROJECT Cobble Mountain Dam

DATE 11/15/79

PROJECT FEATURE Spillway

NAME Tighe &amp; Bond party

DISCIPLINE

NAME

AREA EVALUATED	CONDITION
<u>CUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	Concrete abutment top 6.0' above spillway crest. Crest of spillway weir wall is 6.2' ± above sand approach channel
a. Approach Channel	Good. Some minor hairline cracks of surface concrete at 8'-10' spaces
General Condition	No
Loose Rock Overhanging Channel	No
Trees Overhanging Channel	Sand & gravel floor, ledge sides
Floor of Approach Channel	135' long crest
b. Weir and Training Walls	Good. Recent surface repair
General Condition of Concrete	Minor under bridge
Rust or Staining	Minor less than $\frac{1}{2}$ " deep of surface repair plaster at some joints of training walls
Spalling	None
Any Visible Reinforcing	Minor at some joints & cracks
Any Seepage or Efflorescence	Recent $2\frac{1}{2}$ " Ø plastic, 5' above floor Older $3\frac{1}{2}$ " Ø steel, 1' above floor.
Drain Holes	Good. Wall joint sealer sagging in some joints
c. Discharge Channel	None
General Condition	Some up to 5" Ø downstream of bridge
Loose Rock Overhanging Channel	Concrete past bridge. Ledge beyond that
Trees Overhanging Channel	Brush up to 2" Ø more than 100' below bridge to outlet end
Floor of Channel	
Other Obstructions	

## INSPECTION CHECK LIST

PROJECT Cobble Mountain Dam

DATE 11/15/79

PROJECT FEATURE Spillway Bridge

NAME Tighe &amp; Bond party

DISCIPLINE

NAME

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - SERVICE BRIDGE</u>	
a. Super Structure	
Pearings	None - elliptical arch structure
Anchor Bolts	None
Bridge Seat	None
Longitudinal Members	Minor efflorescence & seepage at joints between 3 segments of arch.
Under Side of Deck	
Secondary Bracing	None
Deck	Bituminous concrete paved. Random cracks at 8' +
Drainage System	Subsurface abutment weep holes near spring line
Railings	Concrete in good condition
Expansion Joints	Good
Paint	None
b. Abutment & Piers	
General Condition of Concrete	Good
Alignment of Abutment	Good
Approach to Bridge	Good
Condition of Seat & Backwall	--- weep holes are functioning some seepage

## INSPECTION CHECK LIST

PROJECT Cobble Mountain DamDATE 11/15/79PROJECT FEATURE Power Tunnel Intake TowerNAME Tighe & Bond party

DISCIPLINE \_\_\_\_\_

NAME \_\_\_\_\_

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - CONTROL TOWER</u>	
a. Concrete and Structural	
General Condition	Excellent
Condition of Joints	Good
Spalling	None
Visible Reinforcing	None
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	None
Joint Alignment	No joints evident
Unusual Seepage or Leaks in Gate Chamber	Not observed - it is submerged
Cracks	None
Rusting or Corrosion of Steel	None
b. Mechanical and Electrical	
Air Vents	
Float Wells	In corner of inlet well
Crane Hoist	Fixed stop log hoist in place. Electric drive controlled at powerhouse or filter plan
Elevator	Battery stand by power
Hydraulic System	None
Service Gates	Good. For service gate. Emergency hand pump
Emergency Gates	60" Ø
Lightning Protection System	20' x 20' steel roller gate
Emergency Power System	None
Wiring and Lighting System in Gate Chamber	Battery & hand hydraulic pump
	None

**INSPECTION CHECK LIST**

## ~~SECRET~~ Cobble Mountain Dam

242 11/15/79

**PROJECT FEATURE** Power Tunnel Intake

NAME Tighe & Bond party

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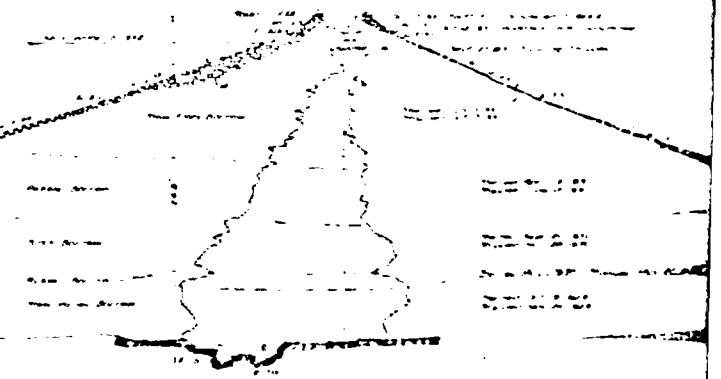
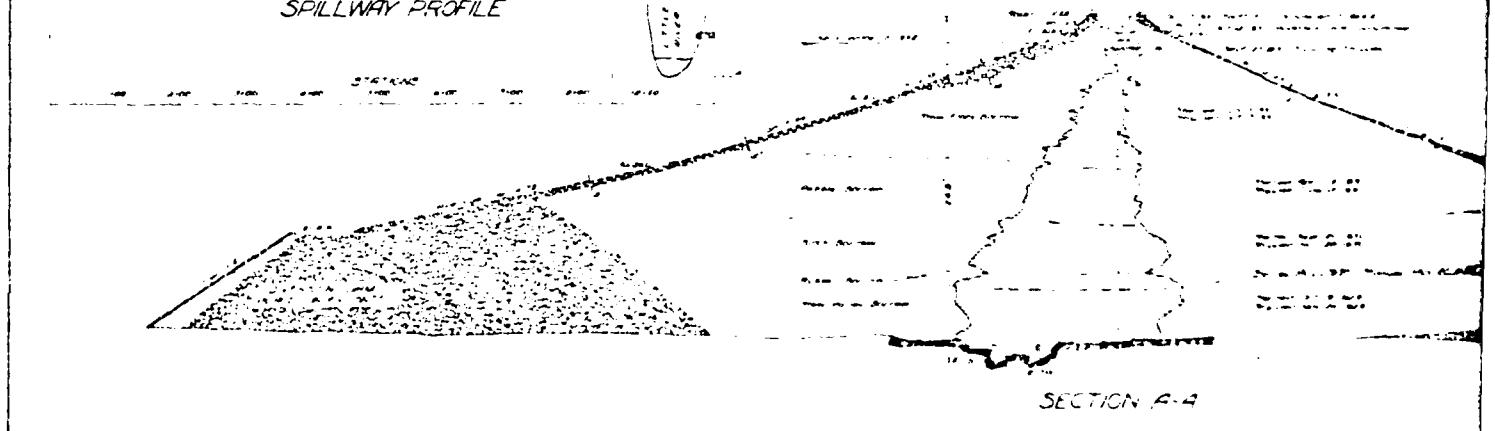
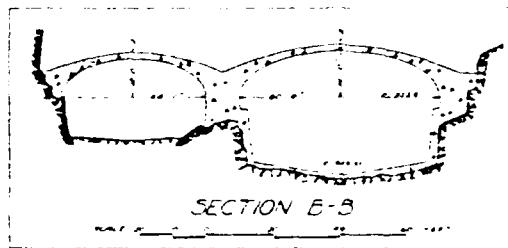
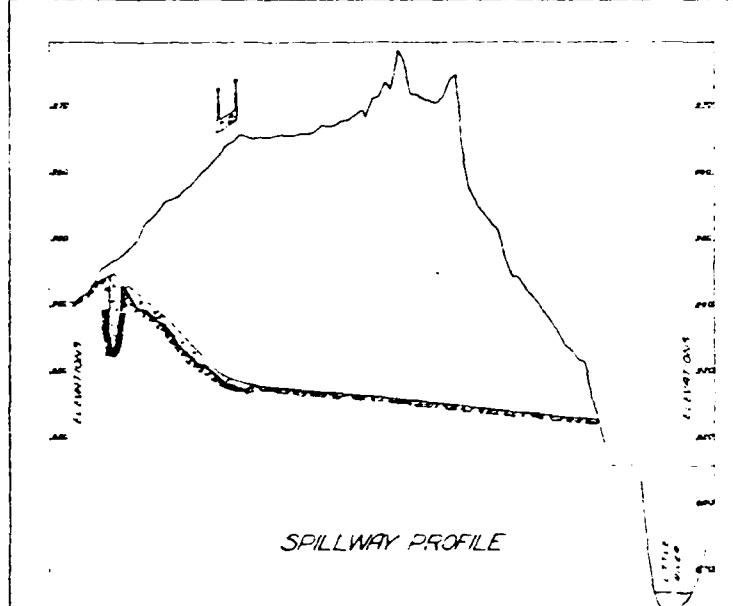
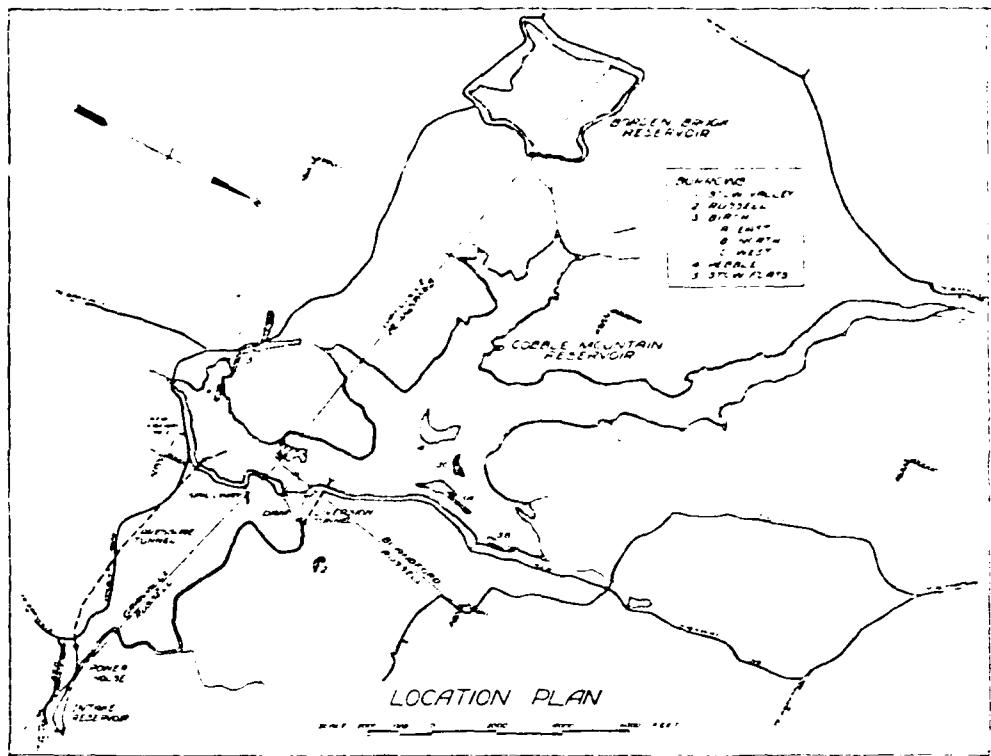
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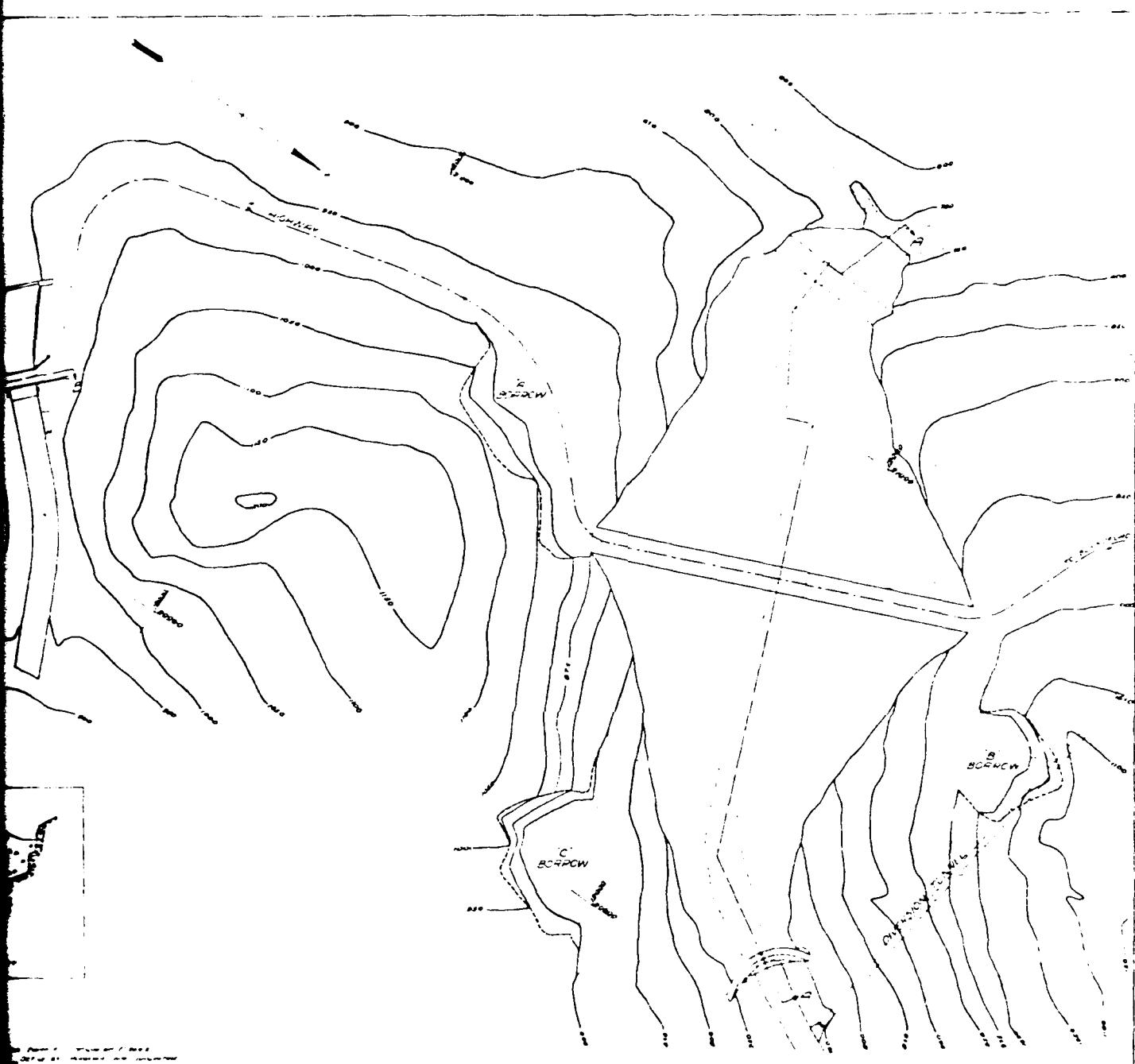
AREA EVALUATED	CONDITION
<u>CUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE</u>	
a. Approach Channel	
Slope Conditions	Submerged
Bottom Conditions	Submerged
Rock Slides or Falls	None evident
Log Boom	None
Debris	None
Condition of Concrete Lining	Submerge. All concrete observed was in excellent condition.
Drains or Weep Holes	
b. Intake Structure	
Condition of Concrete	Excellent
Stop Logs and Slots	Good
Trash screen	3" spaces

APPENDIX B  
ENGINEERING DATA

Design and construction information is available at the following locations:

<u>Item</u>	<u>Location</u>
Construction plans	City of Springfield Water Department
Construction records	City of Springfield Water Department
Report on Construction	New England Water Works Association Vol. XLVI No. 4, December 1932
Notes of James L. Tighe Inspector for Commissioners of Hampden County	Tighe & Bond/SCI, Easthampton, MA
Plans of Spillway Repairs, 1973	City of Springfield Water Department
Inspection Reports	Mr. John Hannon, Mass. D.E.Q.E. Waterways Division, 100 Nashua St. Boston, MA
Plans attached:	
General As Built Plan	Attached hereafter
Design Plans (18 sheets)	Attached hereafter



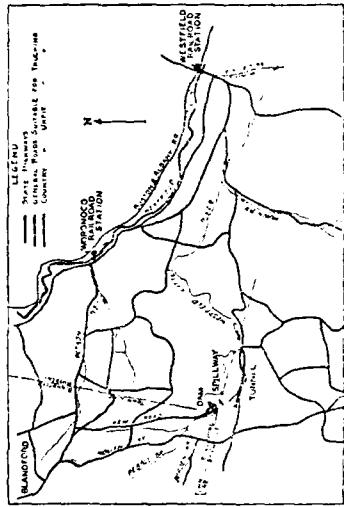


PLAN

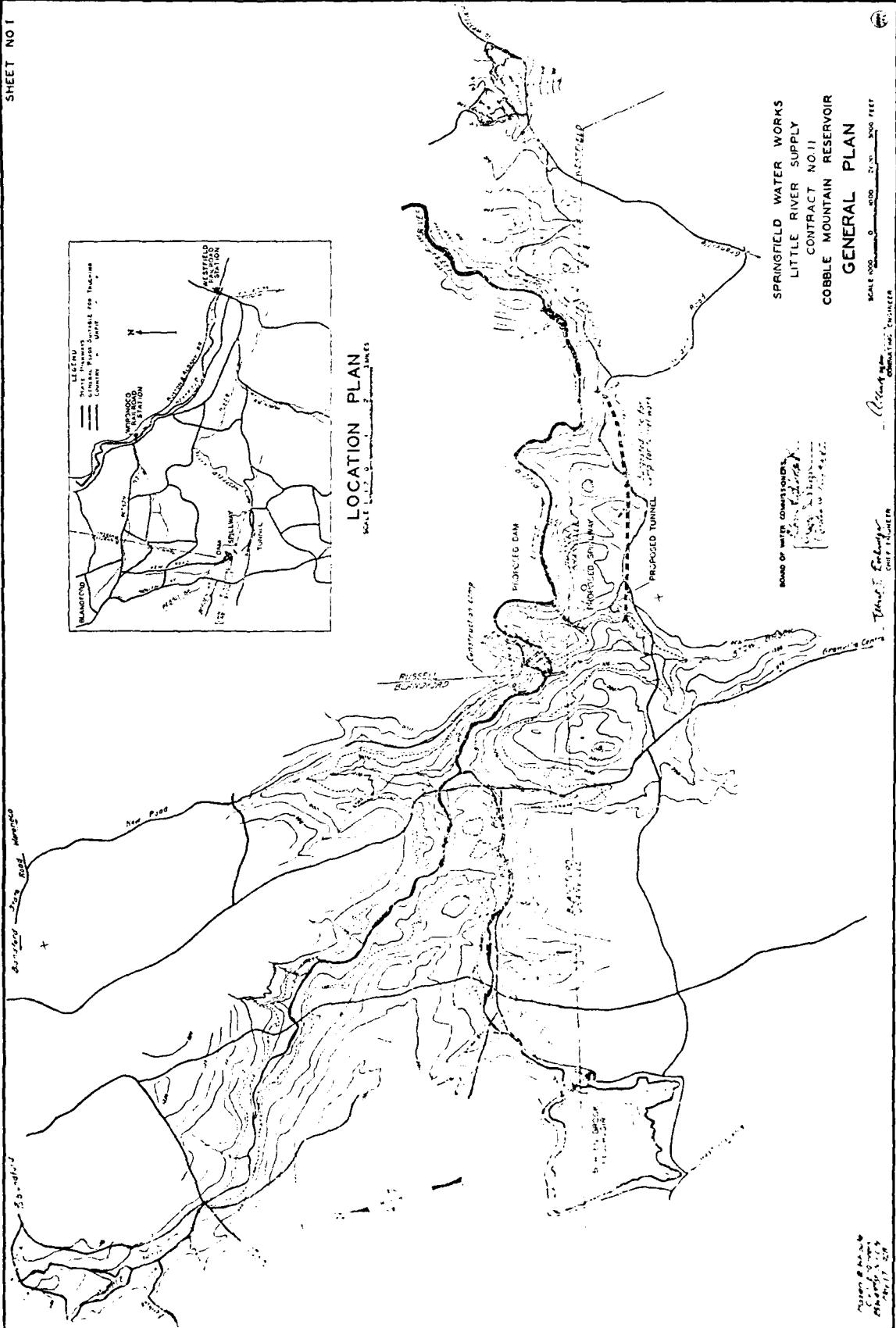
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B-1

SHEET NO 1



## LOCATION PLAN

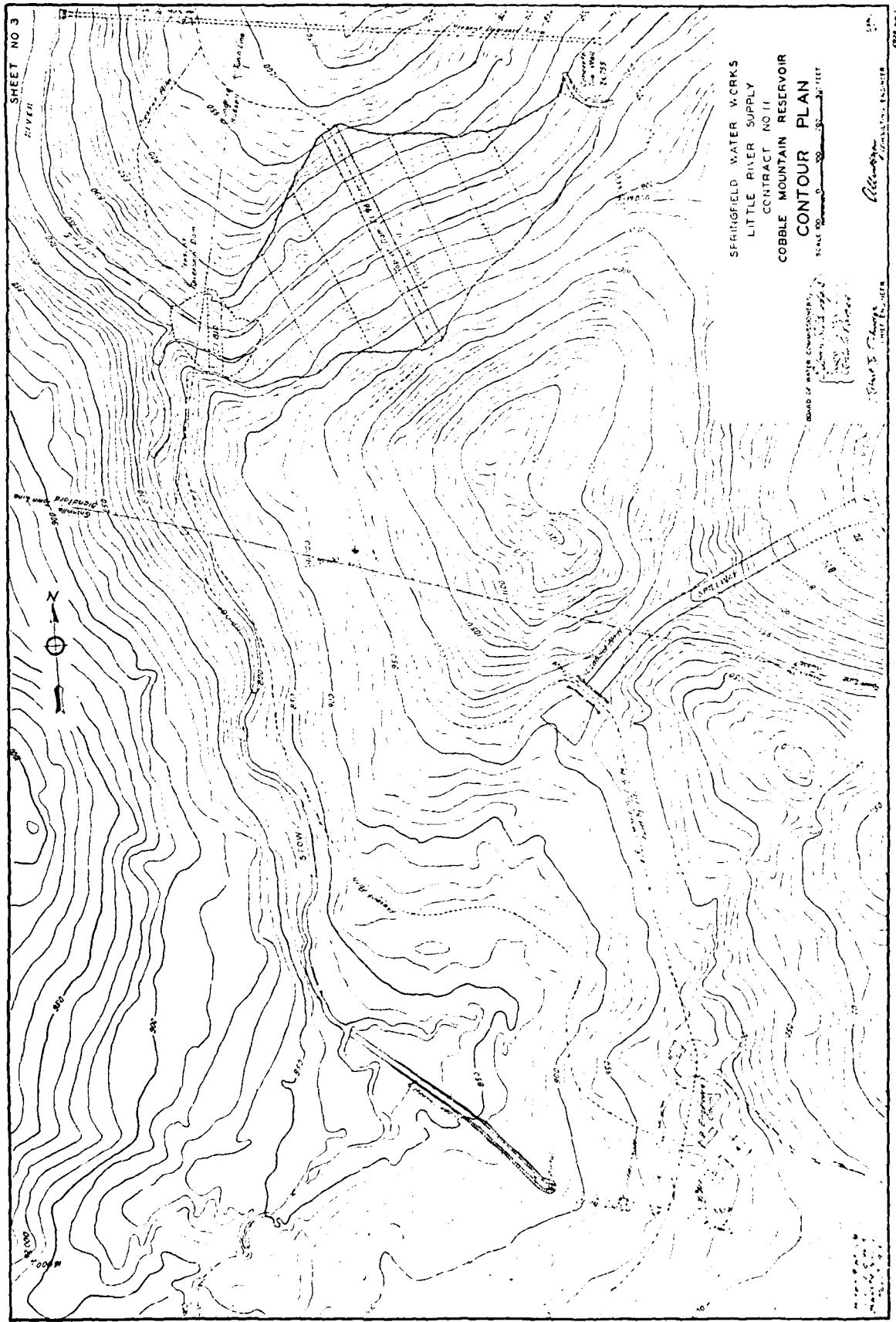


SHEET NO 2

The shaded area is selected as conforming the most suitable and desirable object for dam construction. This area is located in the valley of the Little River, which is a stream of moderate size, and has a much more uniform gradient than the streams which flow through the valley and that would not be desired. The Contractor is not necessarily bound to take over and is otherwise at liberty to negotiate with the owners of the stream and the various property rights holders in the valley. The location of the dam will be determined by the location of the head water of the stream and the method employed from time now to time.

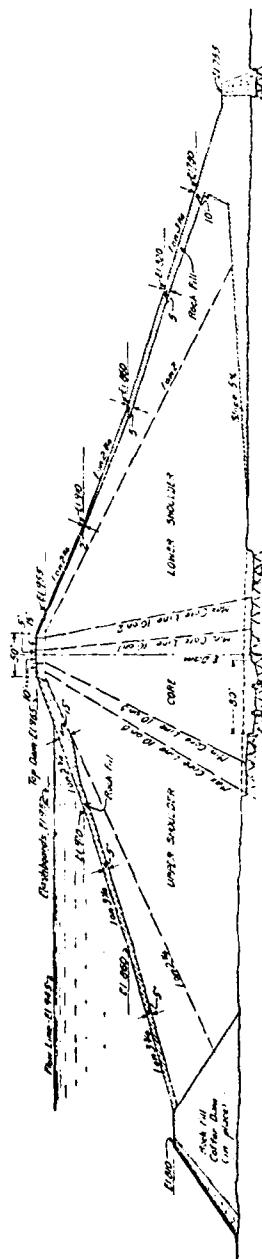


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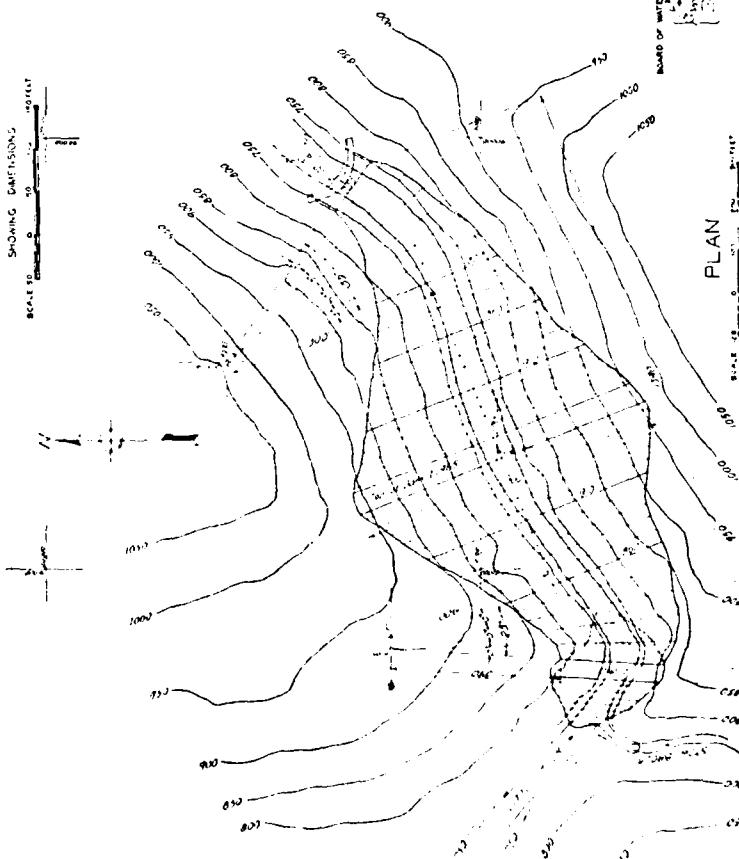


B-4

SHEET NO. 4



STANDARD SECTION  
SHOWING DIMENSIONS  
SCALED TO 1/4 INCH = 10 FEET



1000' 1000' 1000' 1000'

B-5

SPRINGFIELD WATER WORKS  
LITTLE RIVER SUPPLY  
CONTRACT NO. 11  
COBBLE MOUNTAIN RESERVOIR  
DAM  
PLAN AND SECTION  
WITH DIMENSIONS  
SCALES AS LOCATED

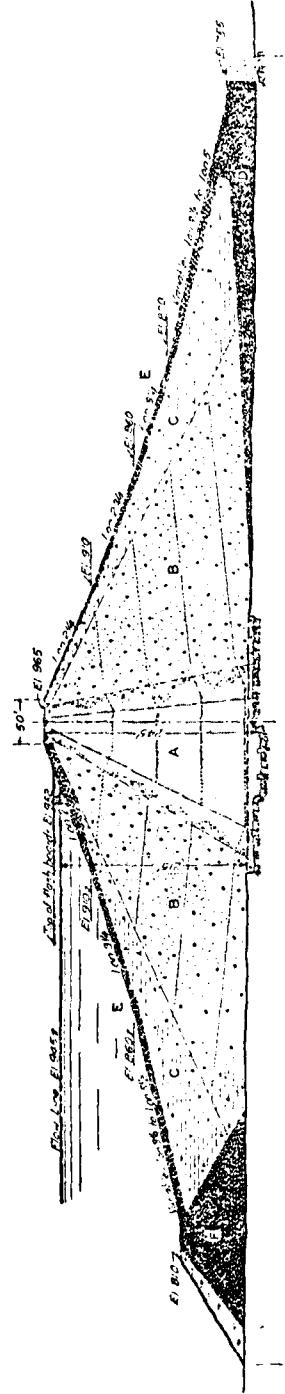
Architectural Engineer

Board of Water Commissioners  
Springfield, Mass.

PLAN

SECTION

1000' 1000' 1000' 1000'

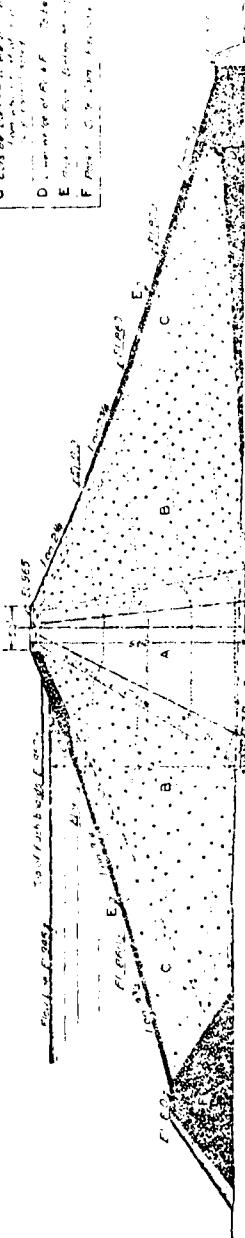


## ACTUAL DEVELOPED SECTION

This diagram shows the actual developed section of the dam following the availability of the rock at the site.

## LEGEND

A	Cored concrete
B	Plain concrete
C	Gravel
D	Rock
E	Rock bottom
F	Plain concrete



## STANDARD SECTION

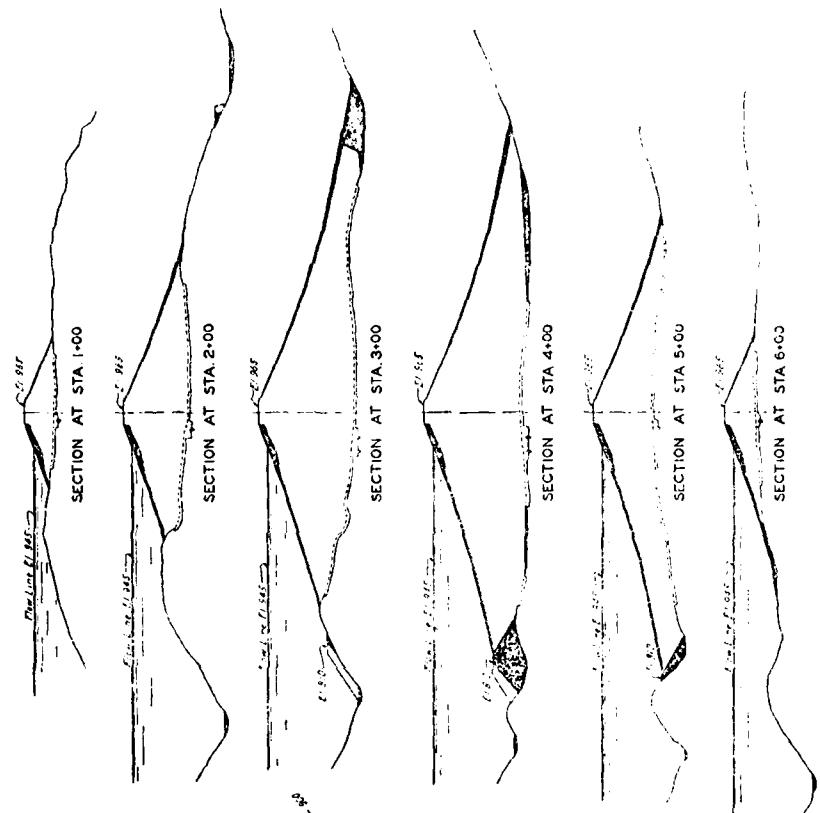
This diagram shows the standard section of the dam. It is based on the actual developed section, but it uses a more uniform thickness for the concrete walls. The sections are divided into three main zones: cored concrete (A), plain concrete (B), and gravel (C). The base of the dam is labeled 'Rock bottom' (E).

SUPERFIELD WATER ACROSS  
LITTLE RIVER CREEK  
CONTRACT NO. 11  
COBBLE MOUNTAIN RESERVOIR  
DAM  
SECTION E

Superfield  
Contract No. 11

B-6

SHEET NO 6



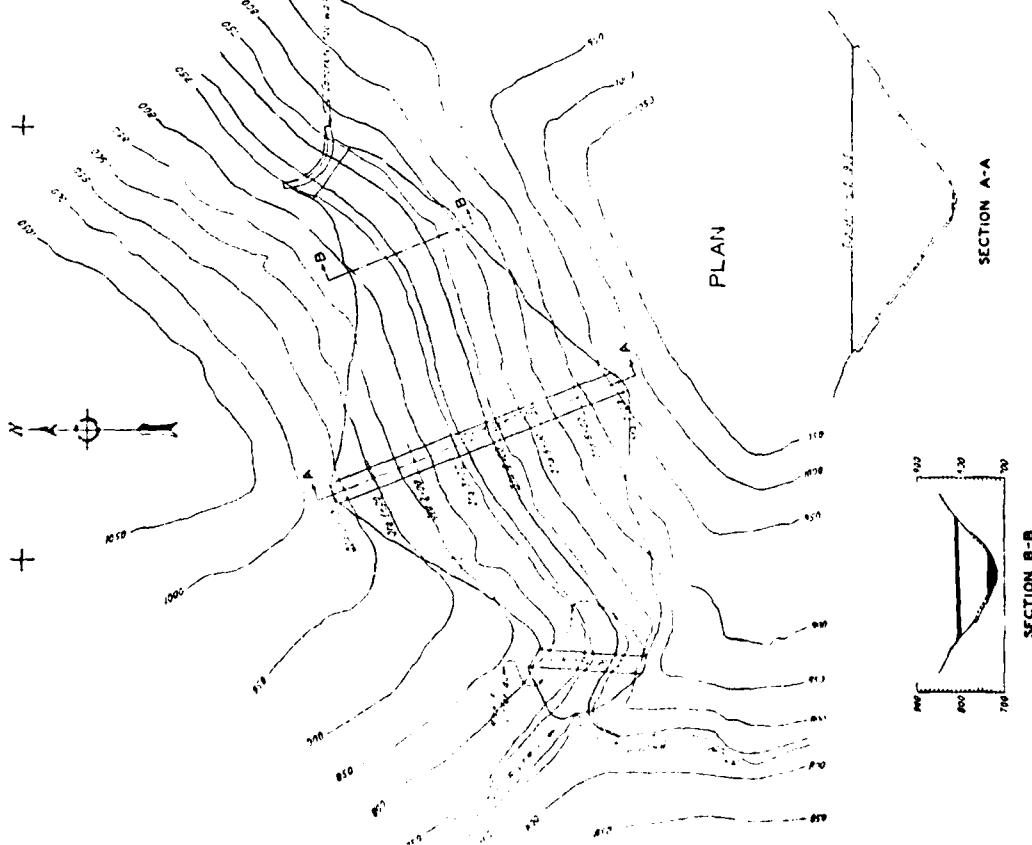
SPRINGFIELD WATER WORKS  
LITTLE RIVER SUPPLY  
CONTRACT NO II  
COBBLE MOUNTAIN RESERVOIR  
DAM

MISCELLANEOUS SECTIONS

Scale 1:20000

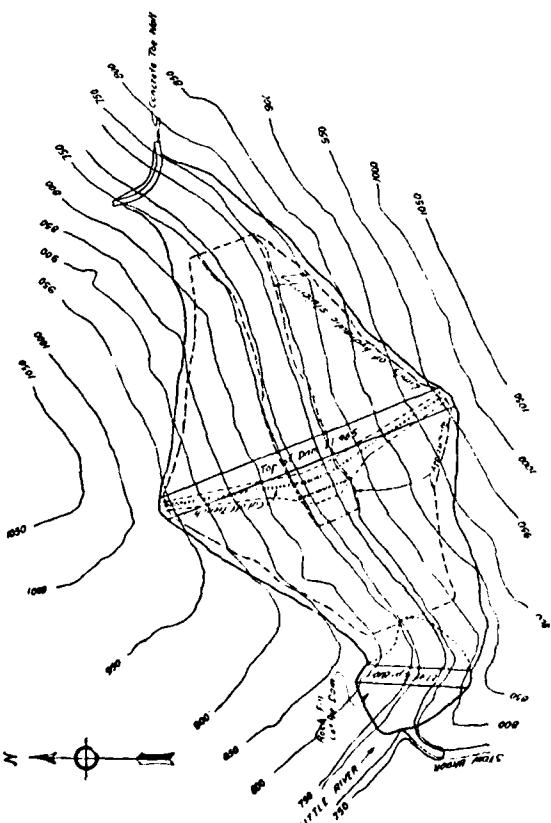
Engineering Drawing  
C. H. Johnson, Chief Engineer

ENR 1-6

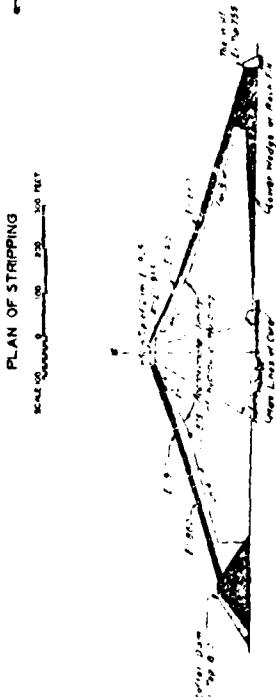


B-7

SHEET NO 7



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SECTION SHOWING LIMITS OF STRIPPING

SECTION E-E

SCALE 0 5 10 15 20 FEET

SECTION D-D

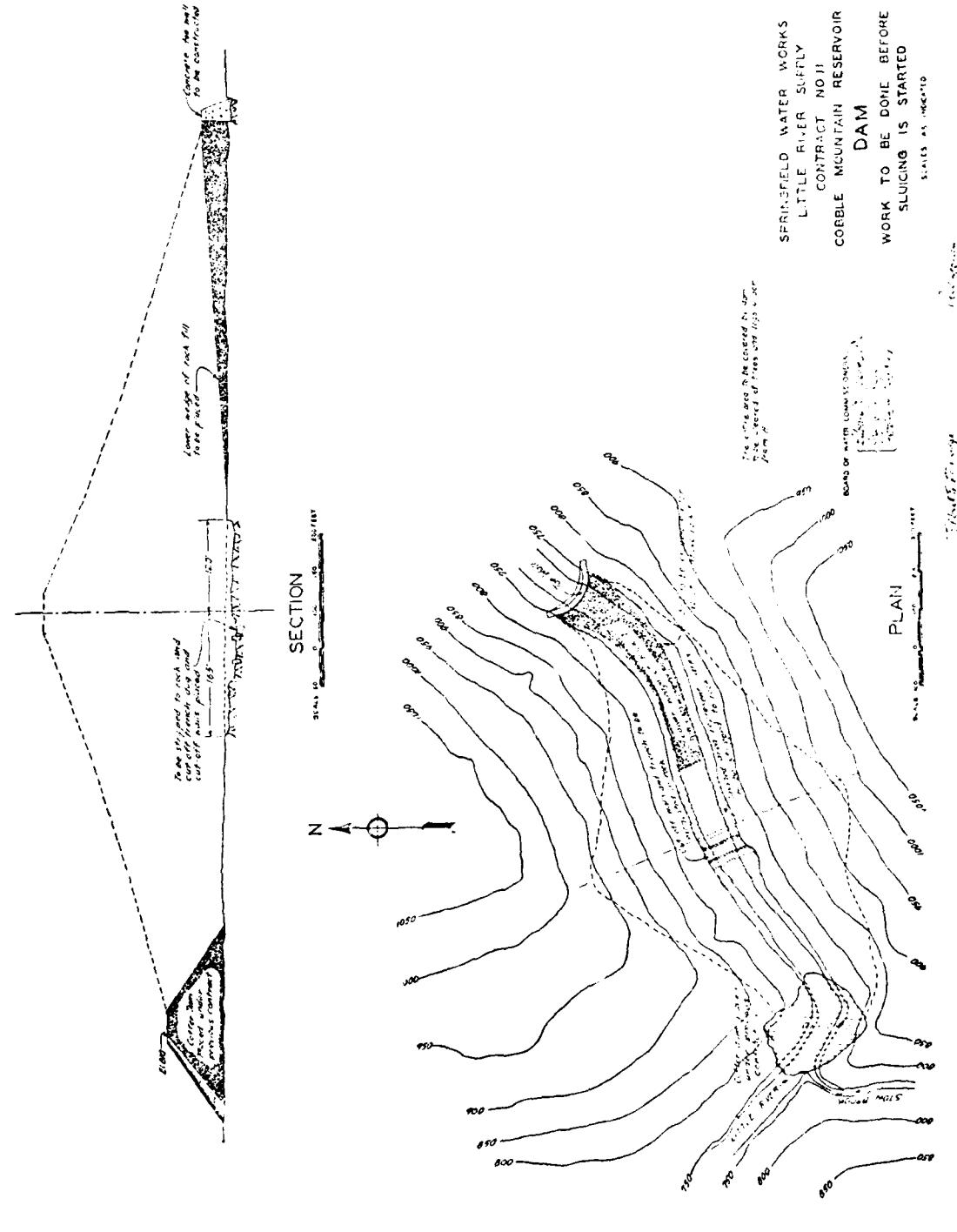
SCALE 0 5 10 15 20 FEET

SUFFIELD WATER WORKS  
LITTLE RIVER SUPPLY  
CONTRACT NO II  
COBBLE MOUNTAIN RESERVOIR  
DAM  
T-OFF TRENCH AND STRIPPING  
KNOB AT INLET-TO

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B-8

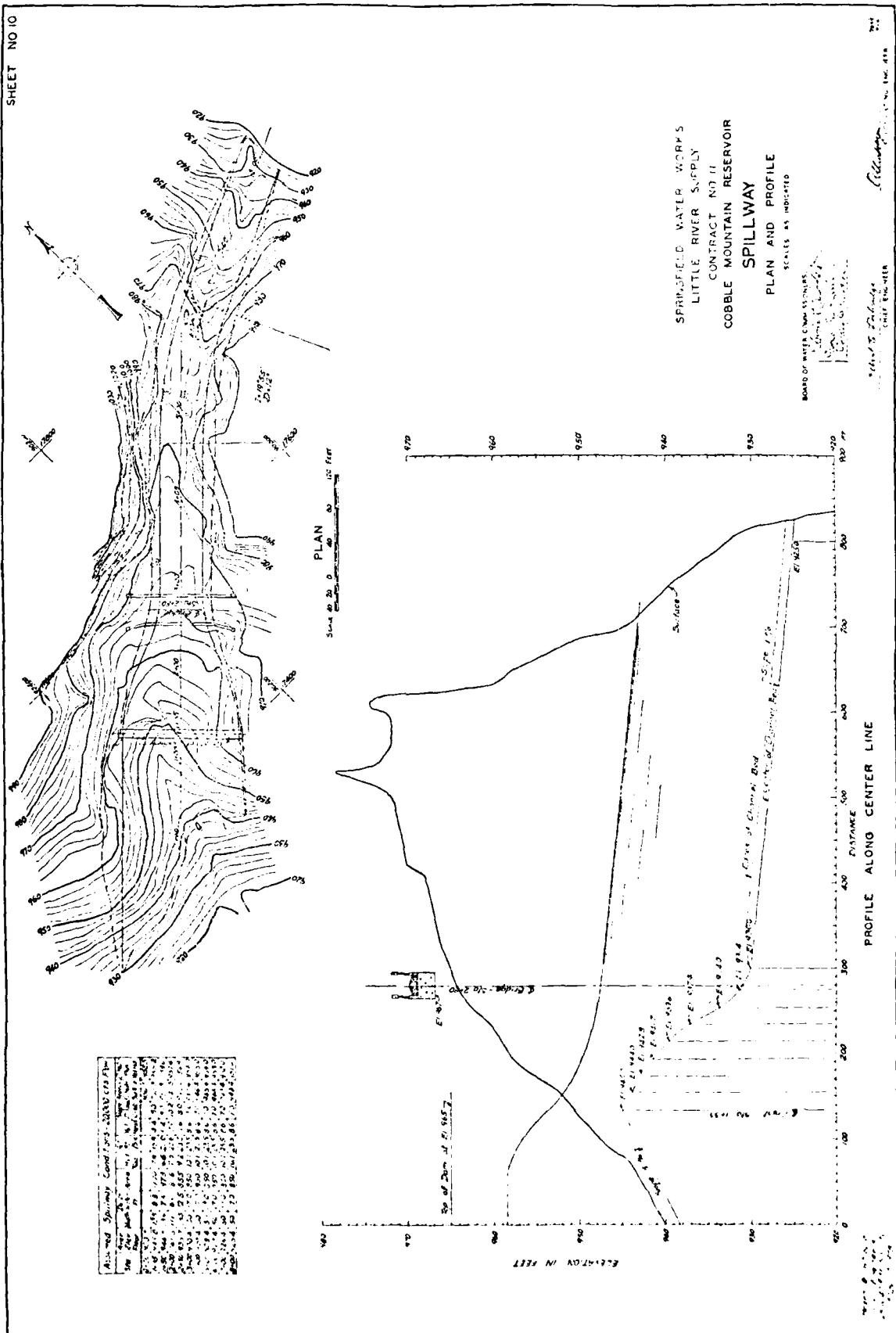
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B - 9

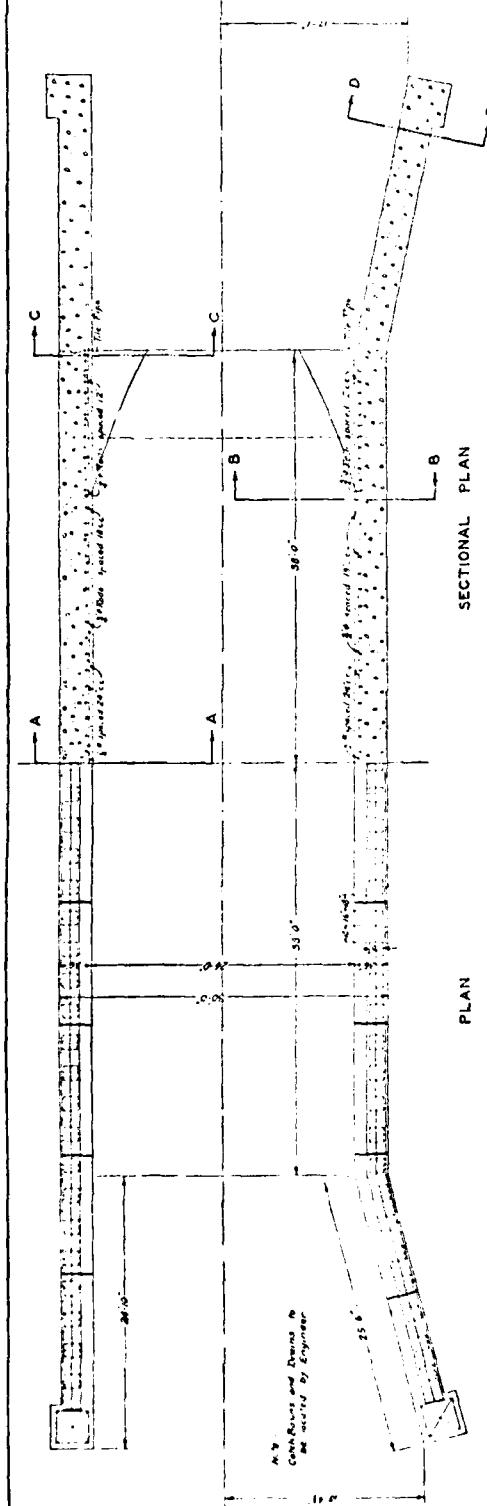


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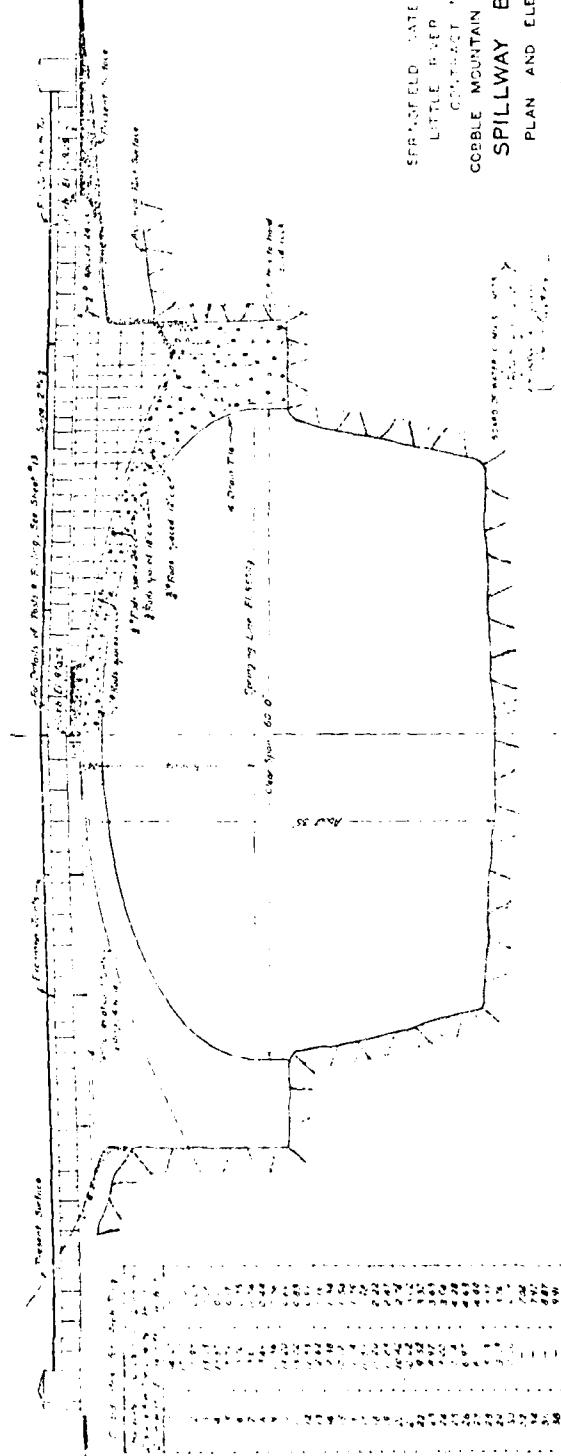


SHEET NO 12



SECTIONAL PLAN

PLAN

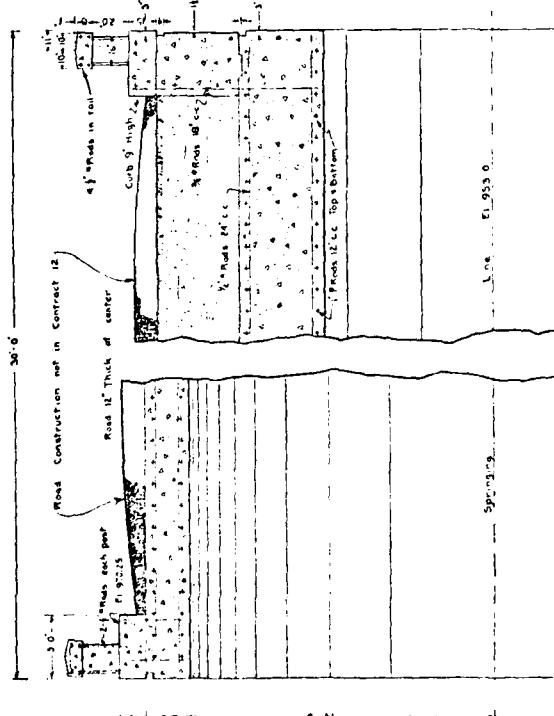


CENTER LINE SECTION

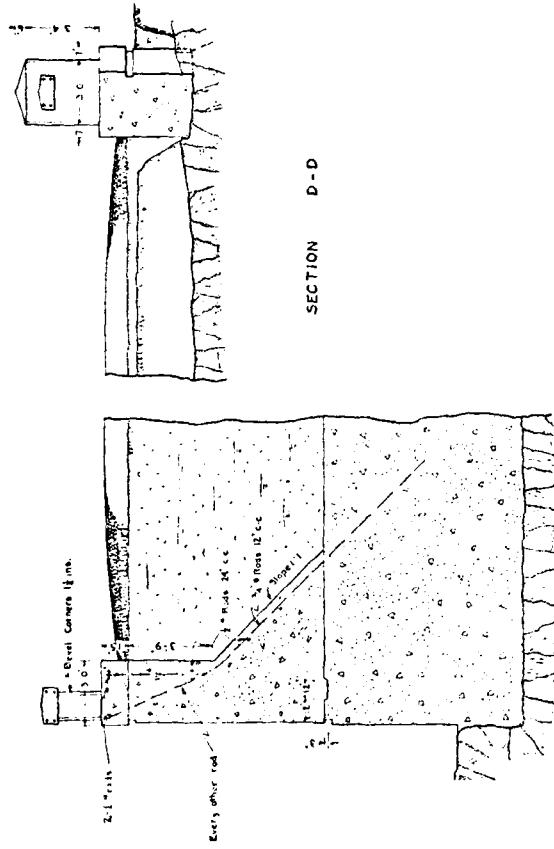
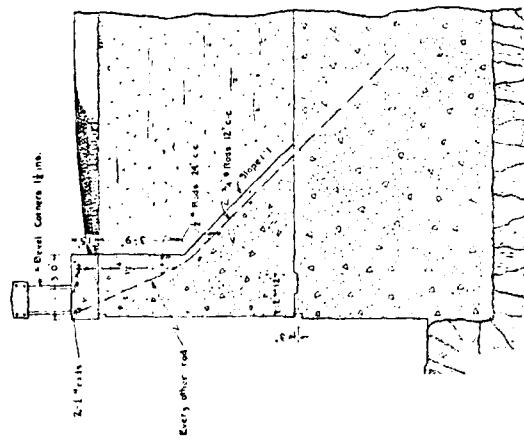
HAI F EI EVALUATION

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B-15



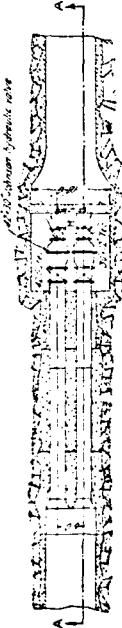
SECTION A-A SECTION B-B SECTION C-C SECTION D-D



SPRINGFIELD WATER WORKS  
LITTLE RIVER SUPPLY  
CONTRACT NO. 11  
COBBLE MOUNTAIN RESERVOIR  
SPILLWAY BRIDGE  
SECTIONS

Master & Johnson  
Architects & Engineers  
Montgomery, N.Y.  
July 7, 1910

B-14



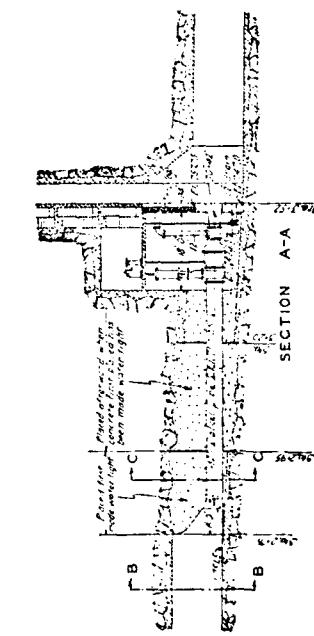
PLAN ON E OF TUNNEL

Note: The date or general idea of the  
original drawings will be used by engineer  
in making changes in case of any change  
in design or any other reason.

SECTION B-B

## LEGEND

- [Symbol: Box] Concrete placed under engineer's control
- [Symbol: Circle] Concrete to be placed under P.M.'s control



SECTION C-C

SHEET NO 14  
SECTION A-A  
SECTION B-B  
SECTION C-C  
SECTION D-D  
SECTION E-E  
SECTION F-F  
SECTION G-G  
SECTION H-H  
SECTION I-I  
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SHEET NO 14  
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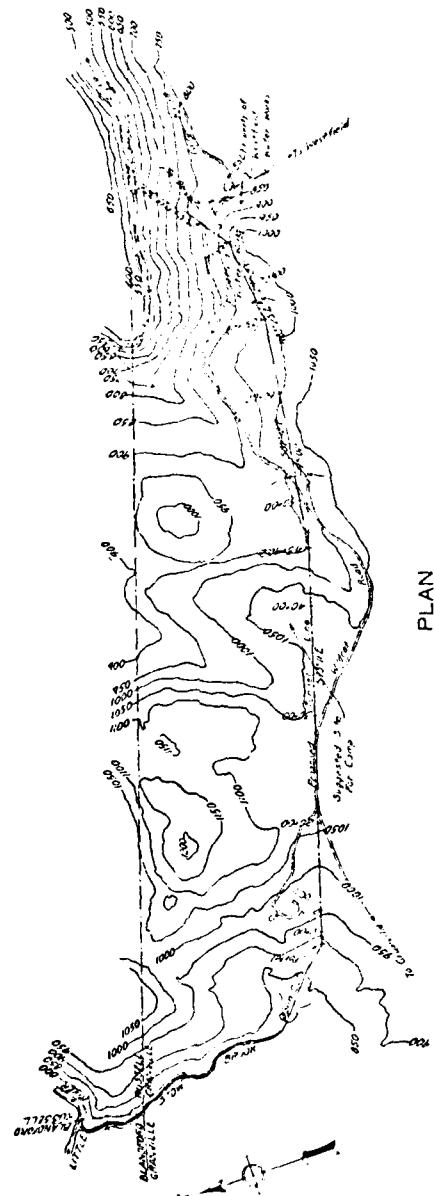
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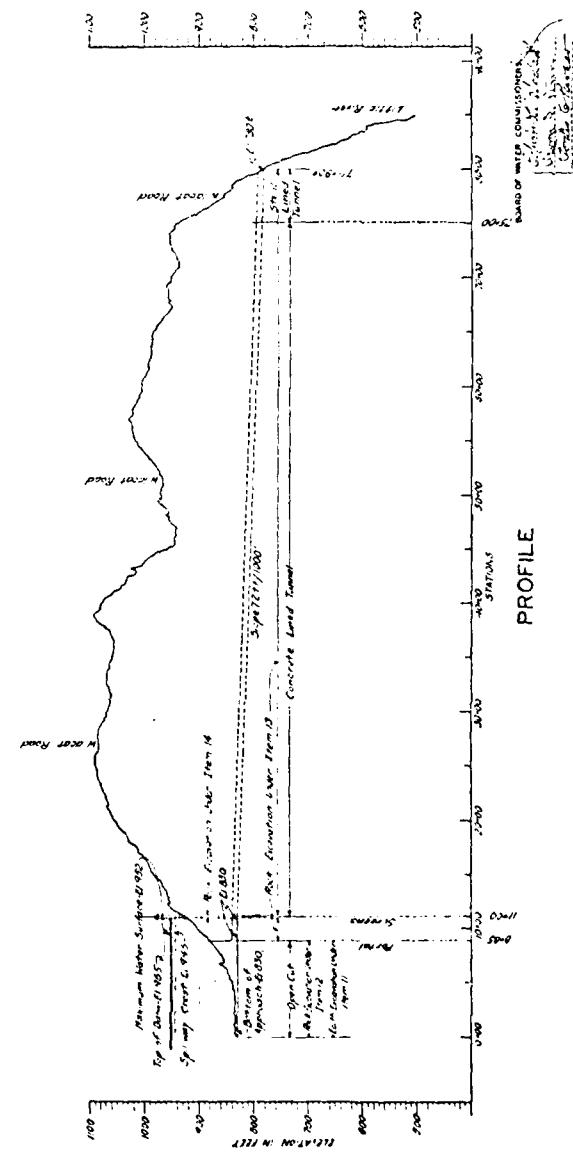
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SHEET NO 15



PLAN

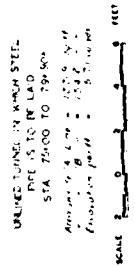
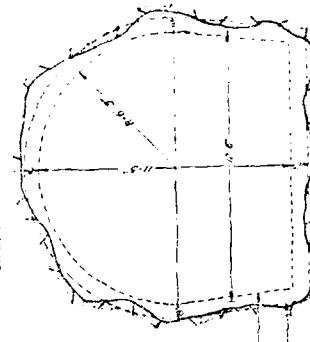
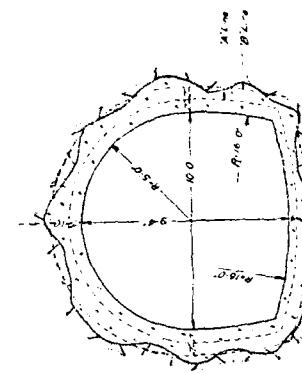
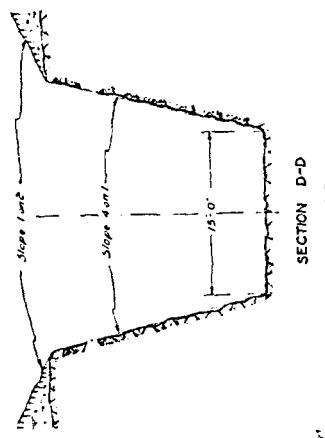
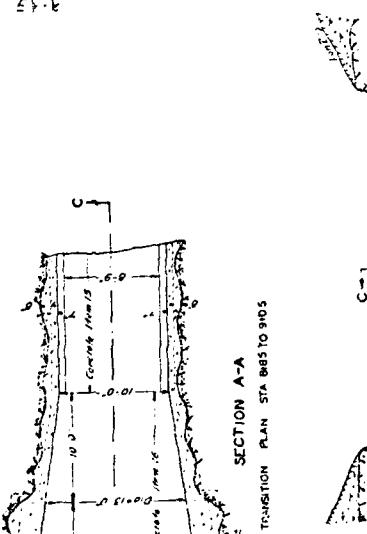
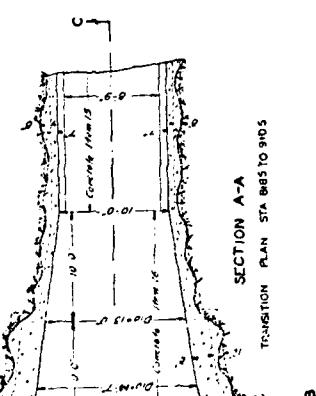
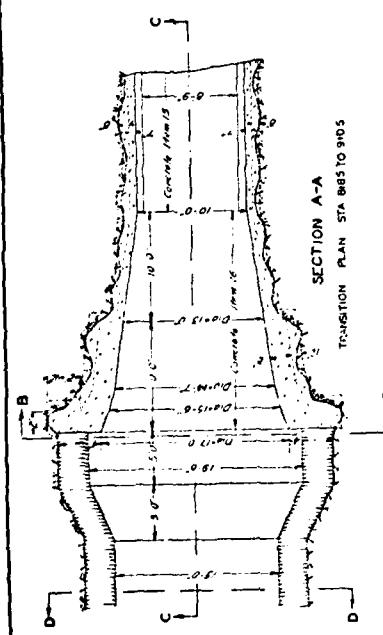


PROFILE

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B-16

SHEET NO 16



SPRINGFIELD WATER WORKS  
LITTLE RIVER SUPPLY  
CONTRACT NO 12  
COBBLE MOUNTAIN TUNNEL  
SECTIONS  
SCALE AS INDICATED

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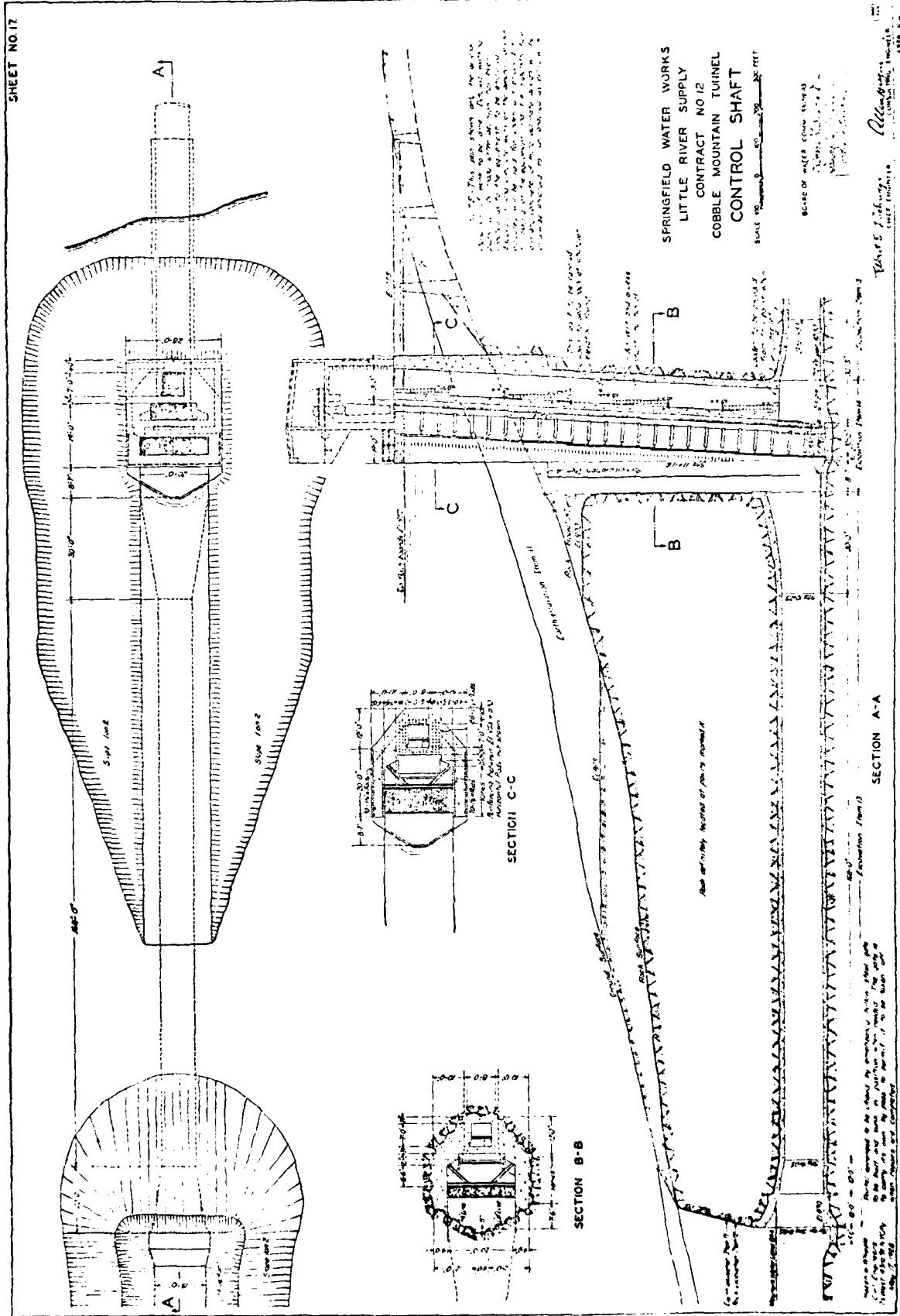
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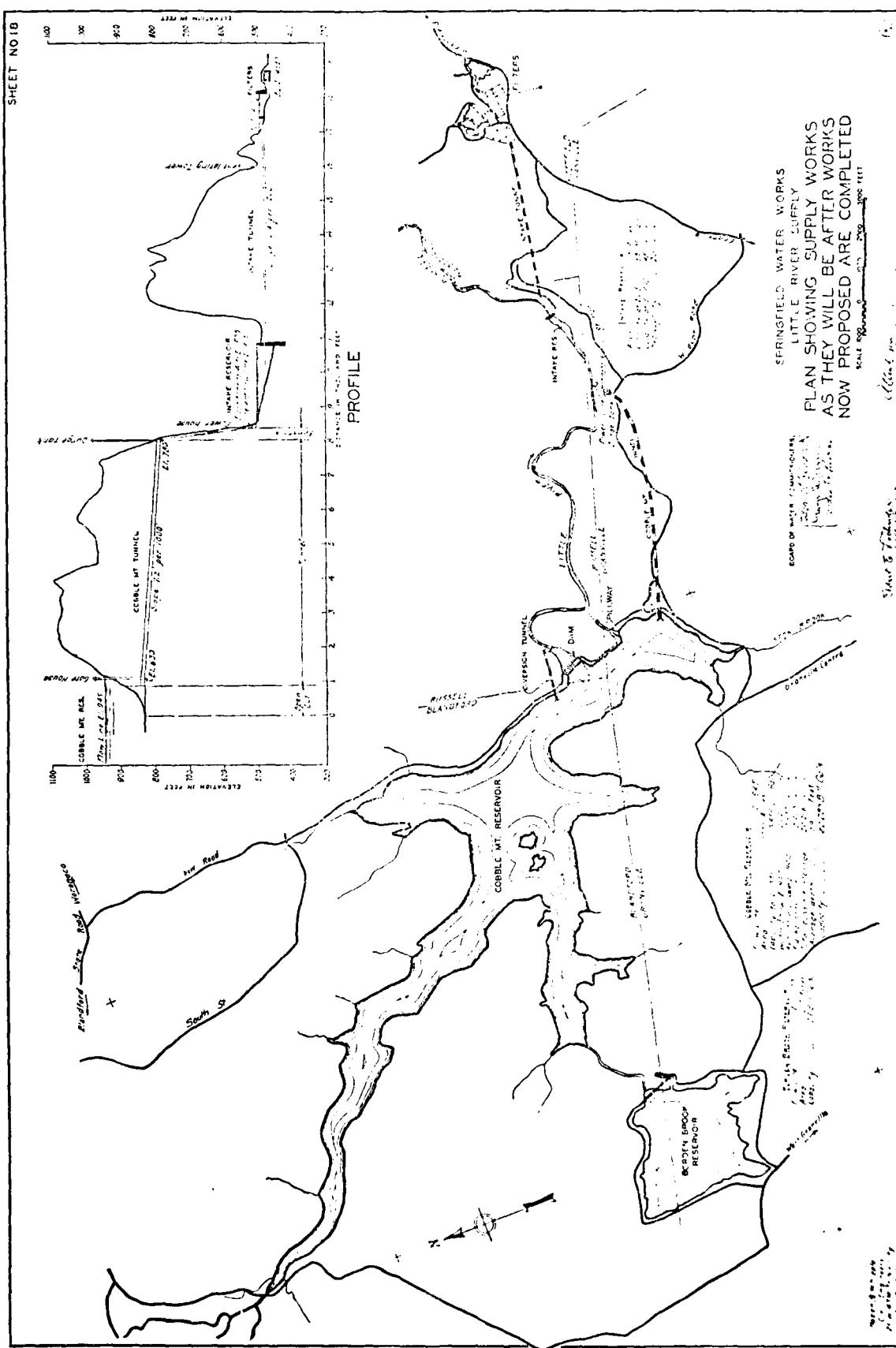
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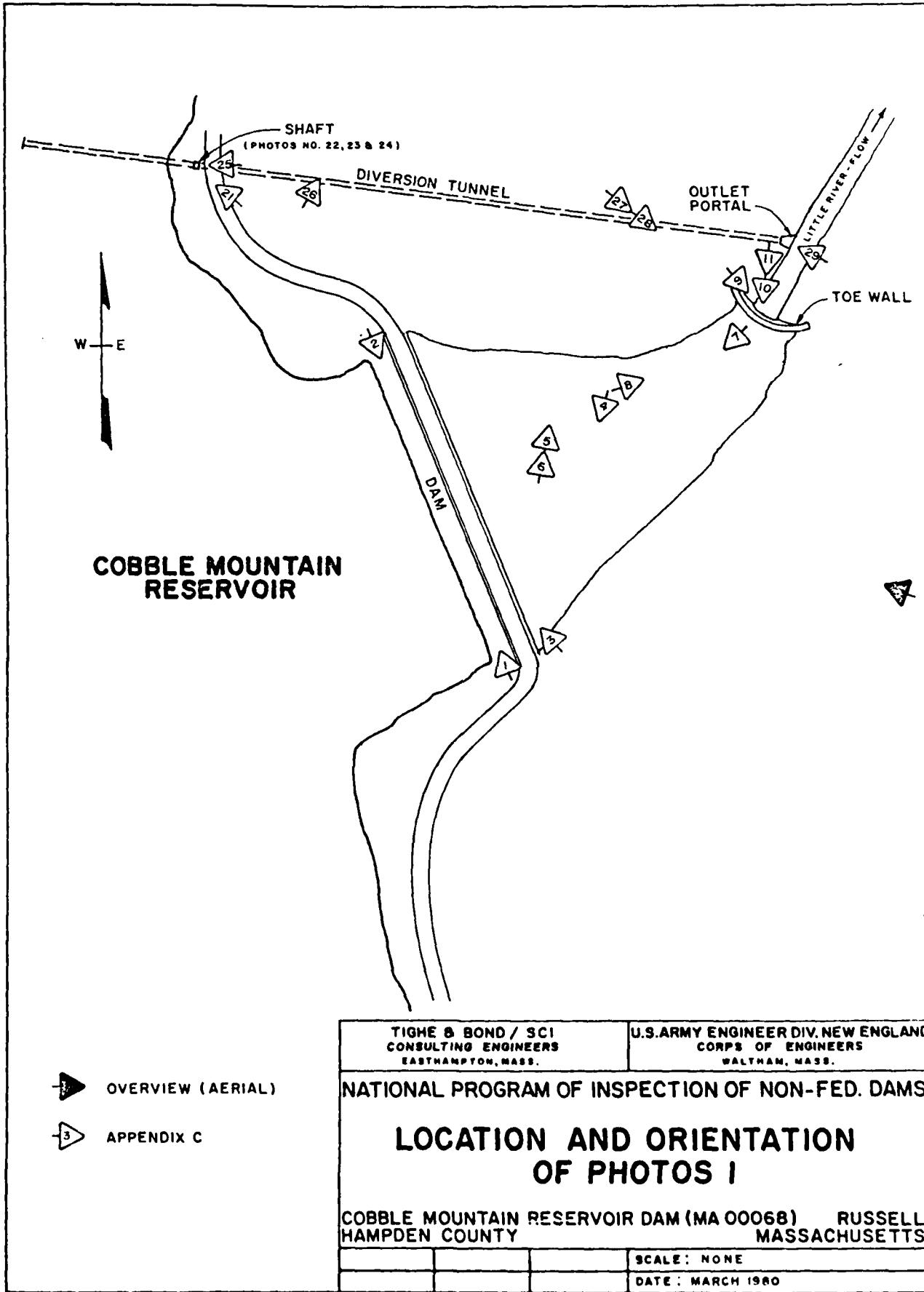
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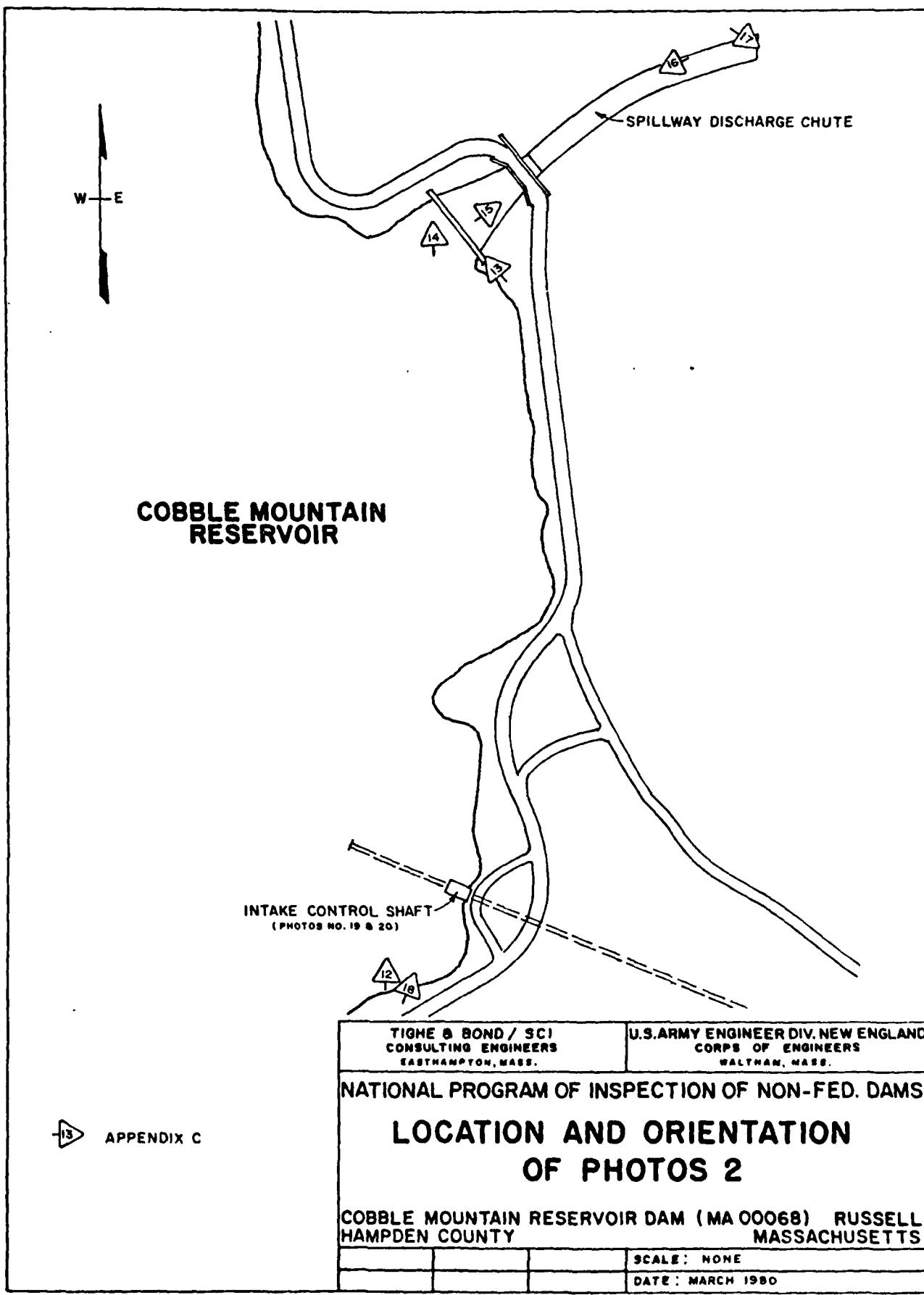


B-18



APPENDIX C  
PHOTOGRAPHS





APPENDIX C



Photo 1

Upstream face of dam,  
looking north at left end



Photo 2

Upstream face of dam,  
looking southeast at  
right end



Photo 3

Close-up view of upstream face of dam,  
highlighting vertical crack or joint

Photo 4

Downstream face of dam,  
upper part, looking south  
at right end



Photo 5

Downstream face of dam,  
mid height, showing rock  
spauls and growth



Photo 6

Downstream face of dam,  
detail of rock surface  
and growth





FIGURE 7

Dominant vegetation at the  
1200 ft. elevation on the south side.

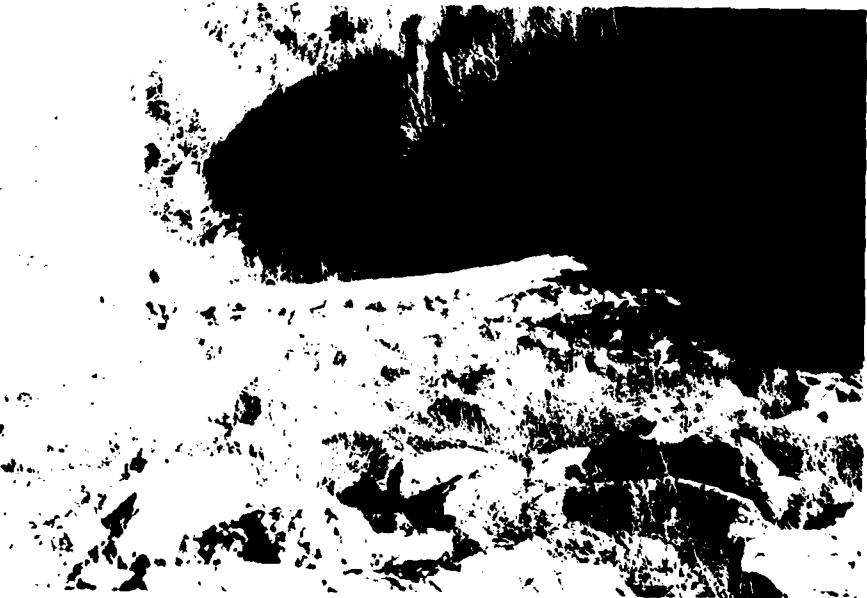


FIG.

1200 ft. elevation on the south side.



FIGURE 8

1200 ft. elevation on the south side.

Photo 10

Downstream face of toe dam  
and weep hole tap



Photo 11

Toe dam weep hole





Photo 12

View looking north from intake structure at spillway; dam is beyond left end of mountain, left of spillway.



Photo 13

Spillway crest wall from right (southeast) abutment.

Photo 14

Northwest end of spillway;  
crest wall showing approach  
channel



Photo 15

Road bridge over spillway  
channel showing repaired  
concrete surfaces & weep holes



Photo 16

Spillway rock cut, looking  
upstream toward bridge





Photo 17

Spillway discharge slope  
from end of rock cut into  
Little River Canyon

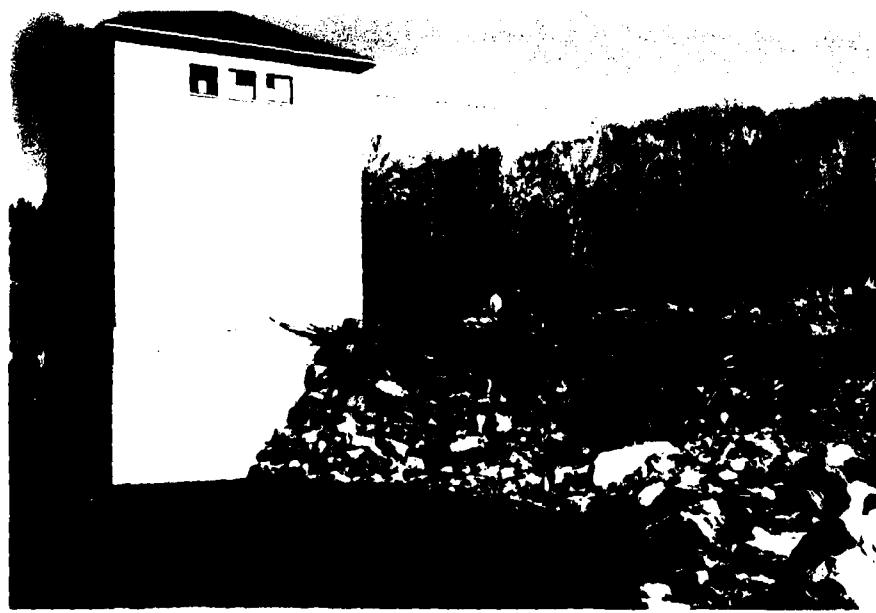


Photo 18

Intake gatehouse from southwest

Photo 19

Maximum reservoir levels  
Aug. 1955, and Oct. 1955;  
at intake gatehouse, showing  
typical good condition



Photo 20

Hand hydraulic pump for  
emergency operation of main  
intake valve at intake gatehouse.





Photo 1

Two-story house with a chimney and several windows.



Photo 2

A person standing next to a large, dark, cylindrical object which appears to be leaking liquid onto the ground.



Photo 3

A group of people standing in a field near a large, dark, cylindrical object which appears to be leaking liquid onto the ground.

Photo 24

Typical operators and biomass  
for drain valves

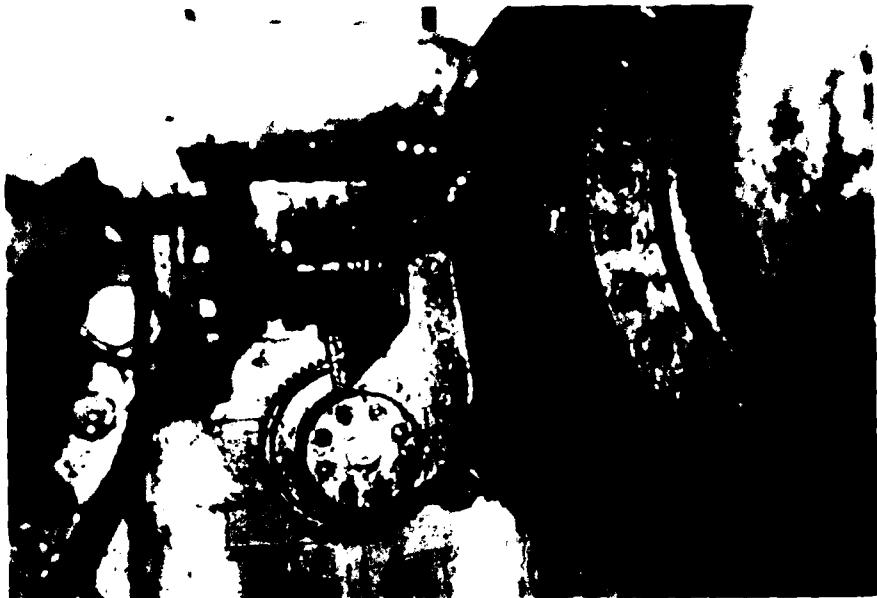


Photo 25

Discharge nozzles of reservoir  
drums in diversion tunnel trans-  
ition chamber



Photo 26

Typical concrete condition in  
diversion tunnel near trans-  
ition chamber





Photo 27

Typical leakage in diversion tunnel ceiling and rock face in last 340 ft. (cont'd.)



Photo 28

Typical leakage near floor of diversion tunnel, in last 340 ft. (cont'd.)

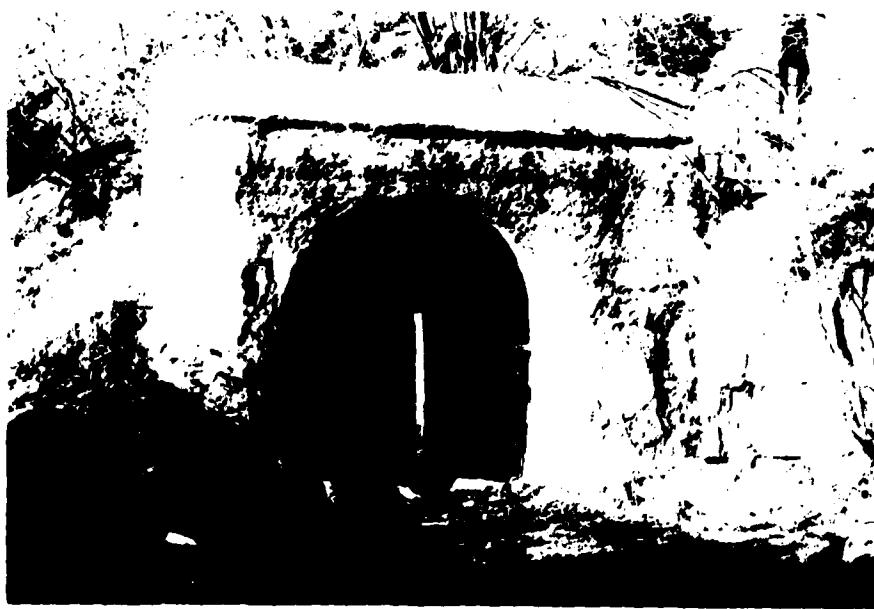


Photo 29

Divergence tunnel, 100 ft. upstream of diversion tunnel.

APPENDIX D  
HYDROLOGIC AND HYDRAULIC COMPUTATIONS

APPENDIX D  
HYDRAULIC AND HYDROLOGIC COMPUTATIONS

INDEX

Spillway Capacity	1-3/10
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SULLIVAN RIVER

At Elevation 950.00 ft. water surface elevation = 950.00 ft.  
 Elevation = 950.00 ft. water surface elevation = 950.00 ft.  
 Headwater ps 156  
 - - - - -

932.0 0

951.0 5.33 1.05

956.0 1.25 4.27

958.0 6.33 7.545 2.0 0

960.0 5.33 12.272 2.3 5.0

962.0 10.33 17.549 1.33 13.1

967.0 15.33 31,640 2.35 4.05

972.0 21.33 56.17 4.05 45.9

975.0 46.91 7.05 10,175 1.43 25.9

For Q ≥ 30,000 cfs water crest will be submerged.

Francis formula if  $C = 1.3 \left[ 1 - \frac{Q}{H + h_v} \right]^{0.55} - 0.52$   $H = C - H^{\frac{3}{2}} \frac{C}{2} = 0.1875$   
 $D = \text{Tailwater depth above weir crest.}$   
 $h_v = \left[ Q_{\text{trial}} / 160(6.5 + H) \right]^2 / 2g$

Elev.	H	$h_v$	D	C	Q	$H_{AE}$	$Q_{\text{obs}}$	$Q_{\text{trial}}$
967.0	15.33	1.73	2.0	3.55	31,640	7.33	5,170	37,010
970.0	18.33	2.37	5.8	3.14	39,920	12.33	7,430	47,350
973.0	21.33	2.69	8.5	3.04	48,220	15.33	3,760	57,920
976.0	24.33	3.14	12.0	2.90	56,460	18.33	12,140	68,630

Spillway capacity

Mile. Cincinnati-Dunn 2.

Spillway number below bridge:

$$\text{Width} = 50'$$

$$\text{Slope} = (128.5 - 923.4) / 15.0 = -10$$

$n$ : rough channel with 1.0 sec. Manning's coefficient of friction

$$A = 50 Y + Y^2/4(50)$$

$$P = 50 + 2Y$$

$$R = A/P$$

$$Q = 1.486 \sqrt{A} R^{2/3} S^{1/2} = 3.72 A R^{2/3}$$

Elev. at 3+00 downstream of bridge

$$933.5 \quad Y = 5'; A = 256 \frac{\text{ft}^2}{\text{ft}}; P = 60; R = 42.5; Q = 2550 \text{ cfs}; V = 14.56 \text{ ft/s}$$

$$945.5 \quad Y = 20'; A = 1100 \frac{\text{ft}^2}{\text{ft}}; P = 90'; R = 11.1'; Q = 31,615 \text{ cfs}; V = 14.56 \text{ ft/s}$$

$$958.5 \quad Y = 30'; A = 1725 \frac{\text{ft}^2}{\text{ft}}; P = 1118'; R = 15.4'; Q = 39,760 \text{ cfs}; V = 23.0 \text{ ft/s}$$

$$965.5 \quad Y = 40'; A = 2400 \frac{\text{ft}^2}{\text{ft}}; P = 1325'; R = 13.1'; Q = 51,570 \text{ cfs}; V = 25.7 \text{ ft/s}$$

$$975.5 \quad Y = 50'; A = 3125 \frac{\text{ft}^2}{\text{ft}}; P = 1531'; R = 20.4'; Q = 56,530 \text{ cfs}; V = 27.3 \text{ ft/s}$$

$$952.0 \quad Y = 23.5';$$

60

969 49

952 crest of spillway

Transverse slope 3%  
949 20

Elev

0

10,000 20,000 30,000 40,000 50,000 60,000

cfs.

It looks like this will submerge Cincinnati no later than 39,000 cfs

If  $n = .035$  & width = 55'  $i' = 1.2^\circ$

then @ Water elev.  $i' = 23.3^\circ$

Spiral curve

Water surface elev. 450.5 ft. M.

### Spiral Drains stream of approach 100 ft

$$\text{Flow } A_{\text{drain}} = 945 - 135.5 = 800 \text{ ft}^2$$

$$\text{Water surface elev. } 450.5 \text{ ft. } Q = 167 \cdot 1450 \text{ ft}^3/\text{sec}$$

$$A = 2y^{1/2} + 135y + 135(y+12) + 135 + 2y^{1/2} \quad \begin{array}{l} A = 135 \\ 2y^{1/2} + 165y + 1620 \end{array}$$

$$P = 2.03y + 10 + 12 + 135 + 135 + 2.03y \quad \begin{array}{l} P = 135 \\ y = 412 \end{array}$$

$$R = P +$$

$$Q = 1.496 \cdot 10^{-4} A R^{2/3} S^{1/2} = 22.15 A R^{2/3}$$

$$A = 135y \quad P = 2y + 135 \quad R = P/A \quad h = .014 \quad S = .665$$

$$Q = 1.496 \cdot 10^{-4} A R^{2/3} S^{1/2} = 22.15 A R^{2/3}$$

$$y = 6; A = 315^{1/2}; P = 147; R = 135/6 = 22.5; h = .014; S = .665$$

$$S = .005; Q = 13.965 \text{ ft}^3/\text{sec}; V = 2.74 \text{ ft/sec}$$

$$y = 12; A = 1620^{1/2}; P = 159; R = 135/12 = 11.25; h = 1.17; S = .005; Q = 37.153 \text{ ft}^3/\text{sec}; V = 12.2 \text{ ft/sec}$$

$$y = 4; A = 540^{1/2}; P = 143; R = 135/4 = 33.75; h = .014; S = .005; Q = 35.363 \text{ ft}^3/\text{sec}; V = 6.3 \text{ ft/sec}$$

Conclusions: Flow is 65,000 ft<sup>3</sup>/sec. Water surface elevation is

is not submerged except for backwater from outlet.

But: rock cut hydrostatic water head will be submerged for  $y \geq 6.5$ . i.e.  $\approx 70,000 \text{ cfs}$ .

Water surface at  $y = 6$ :  $A = 315 \text{ ft}^2$

$$S = (Q / (1.496 A R^{2/3}))^2 = 22.15 / (13.965 / 22.5)^{2/3} = 0.014$$

Since  $h = 0.014$ ,  $y = 6.014$  is the minimum elevation.

Thus, the water surface profile is  $y = 6.014 + 0.044 \sqrt{100 - x} = 6.05$ .

Also there will be a transition zone between

the two methods of calculation. This zone is about 100 ft long.

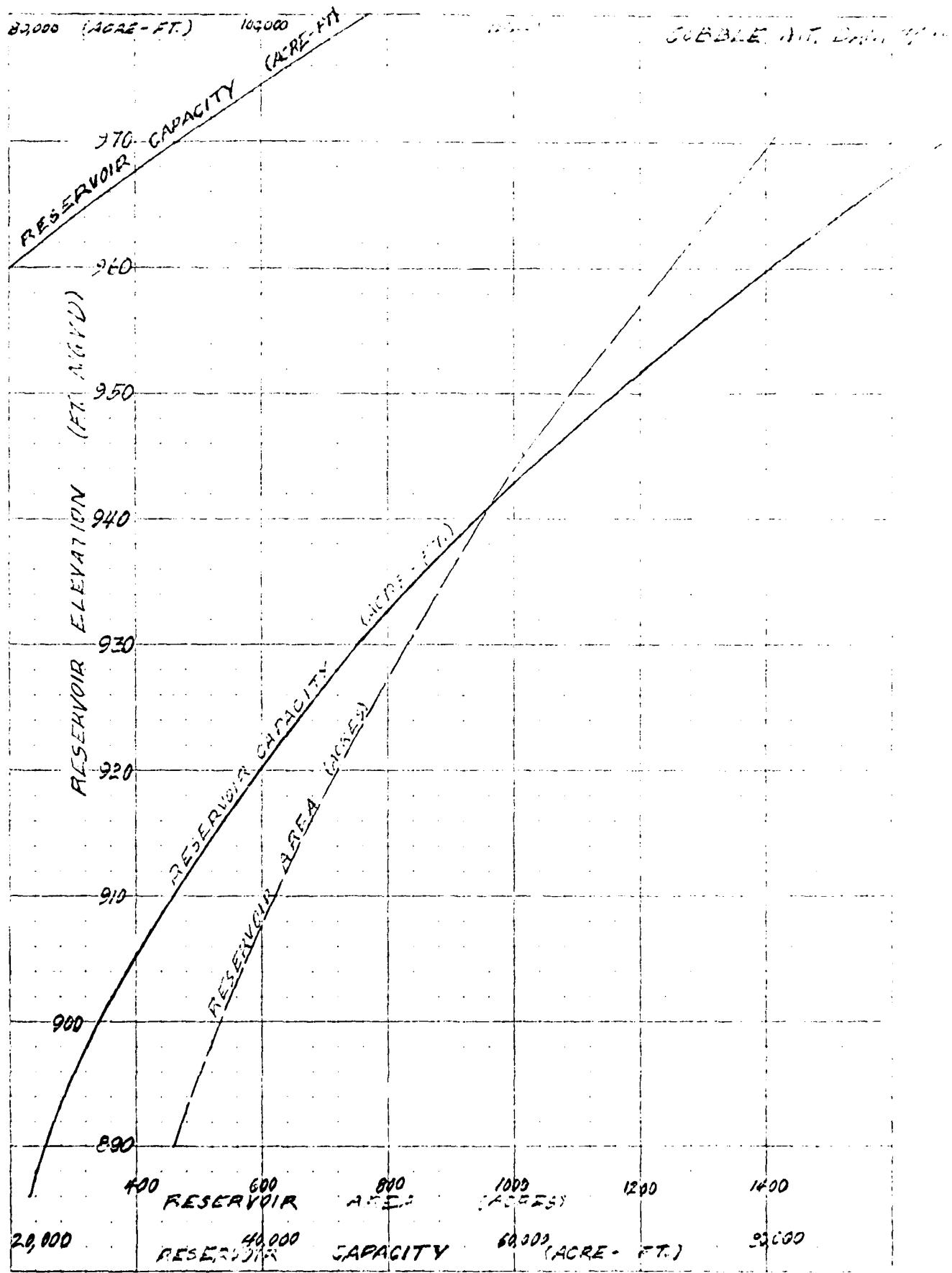
Flow rate in transition zone is  $Q = 167 \cdot 1450 = 240,000 \text{ cfs}$

By Bernoulli:  $h = 167 - 450 = 283 \text{ ft}$

$$y = 8; S = .01; A = 1080^{1/2}; P = 151; R = 7.15; Q = 42,555; V = 39.4 \text{ ft/sec}; h = .015; Q = 38,064 \text{ ft}^3/\text{sec}$$

Add 0.5' to most ent depth to get elev. at spiral outlet msl.

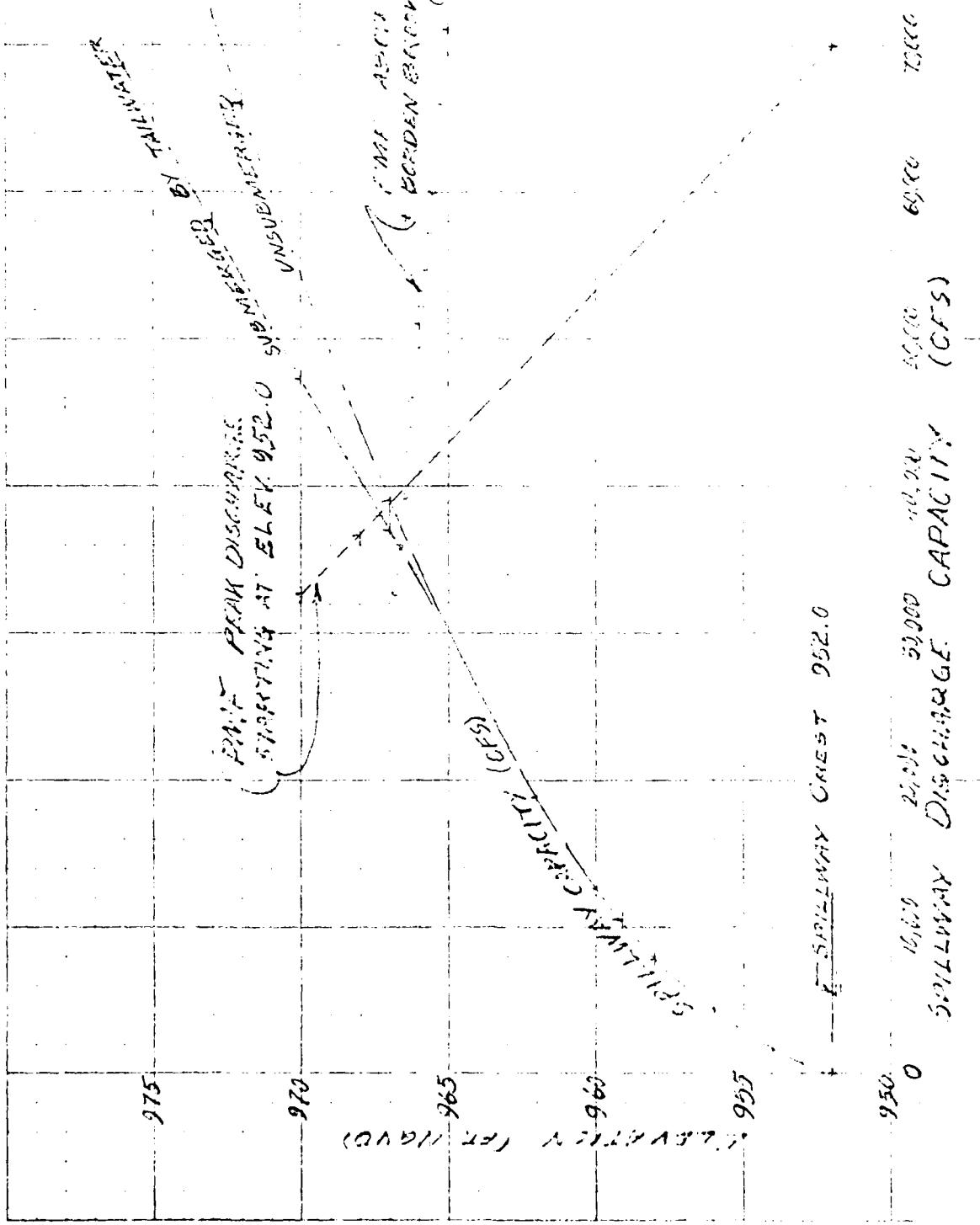
J-3-29



D-4

6-5-29

COBBLE AIT. DANT



D-4A

Spillway Test

Crest Elevation 340 ft 500

Dam size: Height = 972 - 722 = 250' &gt; 100' LITTLE

Impoundment = 90,500 ac-ft &gt; 3,600' LARGE

Hazard Potential: Human: 100+

Industry: 3-4 + Major industrial

Commerce: 2-3, 1-2 shopping areas

7/16/14

Spillway Test Flood Magnitude: P.M.F.

Drainage Area = 45,500 ac-ft

$$P.M.F. \quad Q = 45,500(1+10) = 145,500 \text{ cfs}$$

Elev. ft.s.m.	Spillway Q. Reservoir Capacity ft <sup>3</sup>	Elev. ft.s.m.	Q <sub>1</sub> cfs
975.0	65,000	751,000	41,355
946.0	-	53,000	25,799
960.0	105,000	32,000	6,760
965.0	73,000	36,700	35,444
966.0	33,000	33,000	25,500
952.0	-	70,000	29,600
970.0	47,000	93,000	39,232
968.0	43,000	96,700	37,133

Spillway stage at 975.0 exceeds top of dam - 322' by 2'

Dam would be overtopped by 32.2' by P.M.F.

Route P.M.F. down the reservoir for adequacy

Normal maximum pool = 958.0 = 2-3 sec. at spillway weir

$$Q_{p2T} = 65,950(1 - \frac{6.96}{19}) = 41,794 \text{ cfs}$$

$$Q_{p2T} = 65,950(1 - \frac{2.72}{19}) = 32,265 \text{ cfs}$$

$$Q_{p2T} = 65,950(1 - \frac{0.24}{19}) = 30,495 \text{ cfs}$$

$$Q_{p2} = \underline{31,495 \text{ cfs}} \quad Elevation = 958.5, Freeboard = 6.6'$$

Start P.M.F. at elev. 958.0 = spillway crest

$$Q_{p2T} = 65,950(1 - \frac{6.96}{19}) = 41,794 \text{ cfs}, Elevation = 975.0 \rightarrow 6.6' = 2.4'$$

$$Q_{p2T} = 65,950(1 - \frac{2.72}{19}) = 32,265 \text{ cfs}, Elevation = 960.0$$

$$Q_{p2T} = 65,950(1 - \frac{0.24}{19}) = 30,495 \text{ cfs}, Elevation = 960.0$$

$$Q_{1,2} = 39,131 \text{ cfs}, Elevation = 917.5$$

Elevation = 917.5

D-5

## SPILLWAY TEST

No. 2000 ELEVNT. Dams. E..

## CONCLUSIONS:

PMF = 66,000 cfs.

SPILLWAY STAGE TO MAX PMF 975.2 ft. A.S.L.

Dam overtopped by 3.1 ft.

## SPILLWAY TEST FLOOD ROUTED THRU RESERVOIR

Begin flood storage at base of spillway  
stage 961 ft. Elevation 946.0 ft.

Routed discharge Q = 31,400 cfs.

Reservoir stage 965.5 ft.

Dam Freeboard 1.5 ft.

Begin flood storage at top of spillway 975.2 ft. MSL

Routed discharge Q = 39,000 cfs.

Reservoir stage 987.3 ft. MSL

Dam Freeboard 4.7 ft.

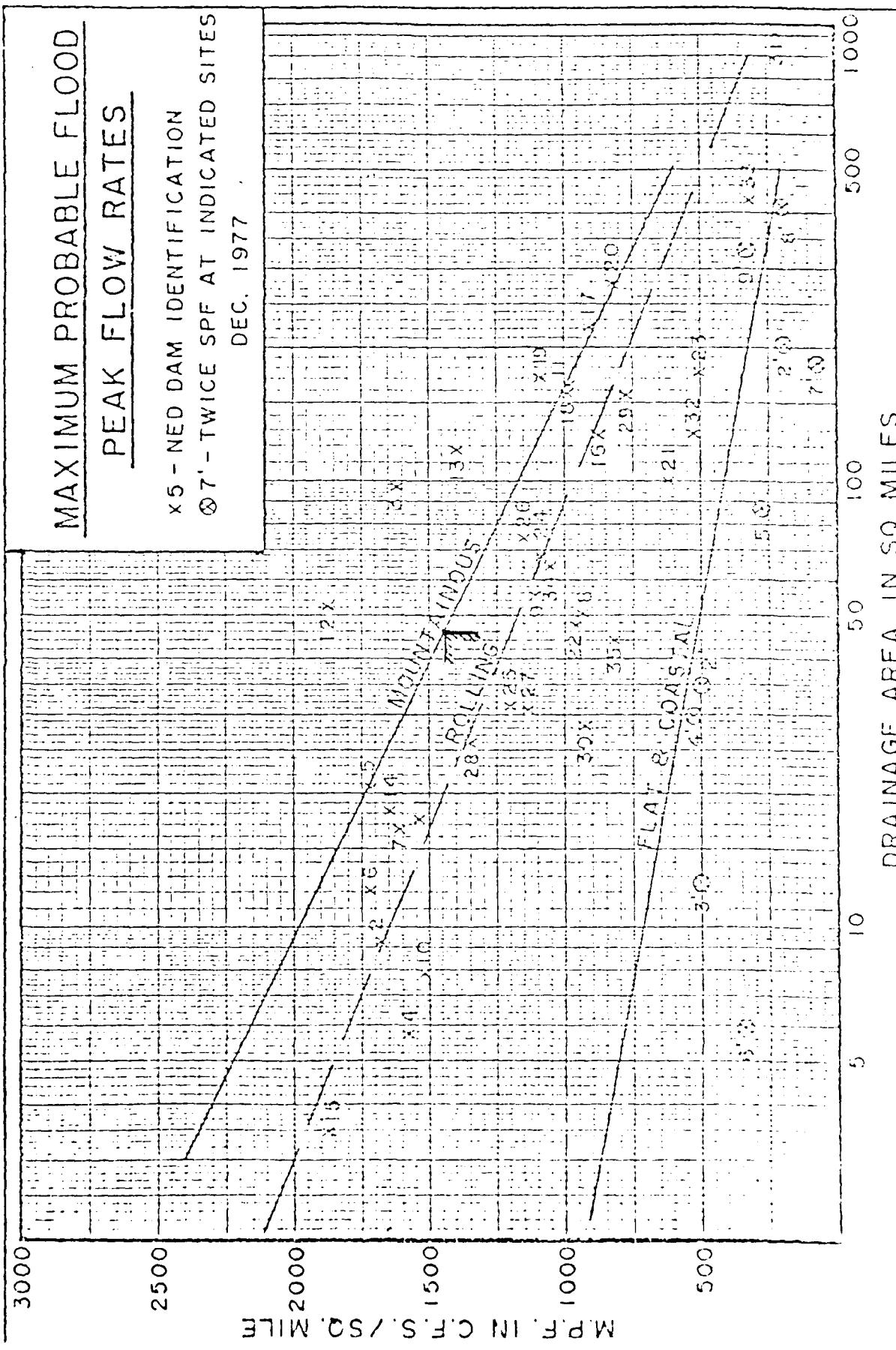
No auxiliary discharge was assumed.

The spillway & reservoir will pass a Probable Maximum Flood without overtopping the dam.  
without auxiliary discharge.

No allowance made for spillway bridges. Some minor  
losses made up by random drawdowns. D-6

MAXIMUM PROBABLE FLOOD  
PEAK FLOW RATES

X5 - NED DAM IDENTIFICATION  
⊗ 7' - TWICE SPF AT INDICATED SITES  
DEC. 1977



AD-A145 564

NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS  
COBBLE MOUNTAIN RESER., (U) CORPS OF ENGINEERS WALTHAM  
MA NEW ENGLAND DIV MAR 80

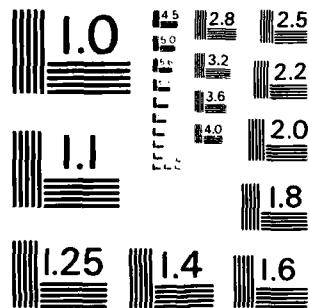
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MICROCOPY RESOLUTION TEST CHART  
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## OUTLET WORKS

Mo. 66-1000 E. 100-134.

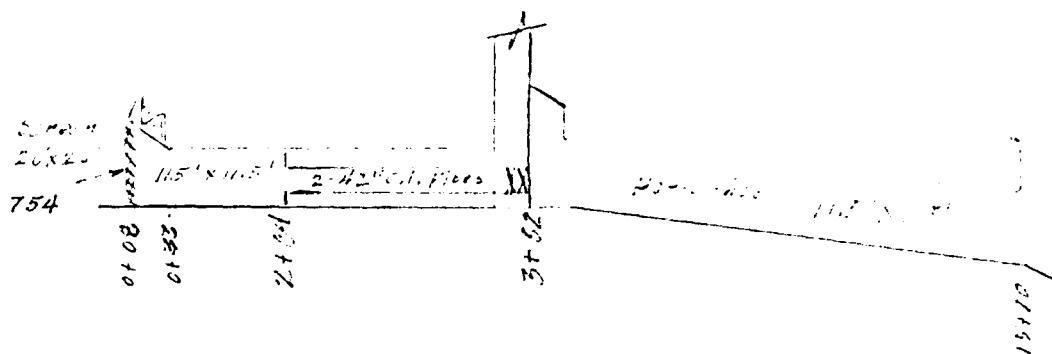
## DIVERSION TUNNEL CAPACITY

Inlet elevation of river

0.0' inlet water level in

ft. 0

750 2



$$H_f = \left( 0.15 \frac{11.5^2}{11.5} + \frac{11.5^2}{2(3.0)} (0.03 \frac{108}{21.5} + 1.2 + 1) + 0.5 \frac{11.5^2}{11.5} + 1 \right) V^2/2g$$

$$= (0.320 + 5.4(4.0 + 1.2 + 1) + 1.5 + 1) V^2/2g = 01.50 V^2/2g$$

$$Q = AV = \pi (11.5/2)^2 \sqrt{2g H / 11.56} = 245.2 \sqrt{H}$$

$$\text{At valve outlet } Q = \pi (11.5/2)^2 \sqrt{2g H / 9.77} = 234.0 \sqrt{H} \quad A = 0.071323$$

Elev. H Q<sub>100</sub>

972.1 242. 3914 TO outlet junction - Top of Dam

972.1 218. 3393 TO valve discharge - Top of Dam

952.0 222. 3653 TO outlet junction - Top of Spillway

946.0 216.0 3604 " - Base of Spillway

967.3 257.3 3777 " - Spillway Top of Flood

CUTLET TUNNEL

1000' CABLE A.M. 2000 ft.

LITTLE TUNNEL CAPACITY

Inlet elevation of invert

83.00

12' height screen  
16' discharge  
concrete

9.33' x 10' maximum section  
concrete lined

83.00

83.5  
20' x 11' screened outlet  
11.5' x 11.5' step top side  
air vent & recessed

730.5 112.2

Storage tank  
Storage tanks  
Reservoir  
Inake reservoir

734.00  
734.20  
734.40  
734.60  
734.80  
735.00  
4.76.2

2 turbines @ 13,650 kilwatts =  
1 " @ 5,250 "

36,600 hp  
7,720  
44,320 hp

Power plant discharge maximum = 1100 + 1000 cfs.  
Head = 952 - 476 = 476 ft.  
Reported by L. M. H. 1916

check head required:

$$H_p = \frac{2W}{500} H / 500$$

$$H = 550 \text{ ft } / 500 = 550 \left( \frac{1.0}{1.0} \right) \left( \frac{1.0}{1.0} \right) \left( \frac{1.0}{1.0} \right) = 419'$$

$$\begin{aligned} \text{Apparent loss in tunnel} &= \text{loss due to } 1.0 - 0.3 = 0.7 \\ H &= \left( 0.15 \frac{230}{11.5} + 0.3 + 0.15 \frac{11.5^2}{9.33 \times 10} \frac{11.5^2}{9.33 \times 10} \right) + 0.5 \frac{730}{500} \frac{11.5^2}{500} = 419' \\ &= 0.30 + 0.3 + 15.1 + 10.3 \frac{11.5^2}{500} = 39.3 \text{ ft } \end{aligned}$$

$$V = Q/A = 1100 \text{ ft }^3/\text{sec} / 1000 \text{ ft }^2 = 1.1 \text{ ft }/ \text{sec}$$

$$H_f = (0.0172)^2 \cdot 50 = 50' \text{ per } 1000 \text{ ft } = 0.05 \text{ ft }/ \text{ft}$$

CALCULATED

WATER LEVELS IN FEET

INFERRED DRAINS - DEPTH

$$Q = \pi R^2 = \pi (2)^2 \times 1.5 = 12.57 \text{ cu ft sec}^{-1}$$

$$A = \pi R^2 = \pi (2)^2 = 12.57 \text{ cu ft sec}^{-1}$$

Flowing water head = 4.66

Bottom elevation = 2.00 (4.66 - 2.66) = 1.66

Top of drain = 3.66 (4.66 - 1.00)

Top of piping = 4.66 (4.66 - 0.00)

Top of dam = 2.72 (4.66 - 1.94)

Top of piping Test 160.1 = 267.3 (4.66 - 1.13)

SPILLWAY TEST

1940 - 1941 DECEMBER 11-12

Borden Brook Reservoir

$$\text{Normal capacity at top of dam} = \\ 2,500 \text{ ac. ft.} = 7,073 \text{ cu. ft.}$$

$$\text{Crest Mt. Capacity at } 432.0 = 7,073 \text{ cu. ft.}$$

$$\text{Crest Mt. Cap. at } \frac{958.0}{6.0} = 77,573 \text{ cu. ft.} \\ \text{stage increase to storage} = 6.0'$$

Check ration = PMF + Elevation

Since Elevation does not change, PMF does not change.

$$Q = \text{PMF} = 65,950 \text{ cfs}; \text{Runoff volume} = 19 + 25 \times 65,950 = 1,000,000 \text{ cu. ft.} \\ 7073 \text{ cu. ft.} = 3,14 \text{ sec. of time required.}$$

$$Q_{p27} = 65,950 \left(1 - \frac{9.57}{14-3}\right) = 37,130 \text{ cfs; Elav. } 270$$

$$Q_{f27} = 173,200 \left(1 - \frac{9.47}{22}\right) = 42,500 \text{ cfs; Elav. } 950$$

Spillway capacity at Elav. 950 = 49,600 cfs

Check Borden Brook Reservoir Dam failure

$$\text{Height} = 1070 - 1000 + 10 \pm = 90'$$

Length reported to be = 720'

$$\text{Length after } 1000' = 720/2 = 360' = 360 \pm \text{ width from 720'}$$

Capacity at top of dam.

$$10(213 + 2.51)/2$$

$$7,073 \text{ cu. ft.}$$

$$\underline{2352}$$

$$23,933 \text{ cu. ft.}$$

$$Q_{p1} = \frac{9}{27} \cdot 46 \sqrt{g} \cdot h^{3/2}$$

$$= \frac{9}{27} \cdot 46 \cdot 360 \sqrt{g} \cdot 80^{3/2} = 173,200 \text{ cfs}$$

Borden Brook water shed = 3.1 sq. mi. vs. 45.3 sq. mi.

Borden Brook PMF peak would be earlier than visible 1/2 hr.

Start Borden Brook failure at 8' above 1000' = 3,14 \cdot 260 = 1955

$$Q_{p27} = 13,000 + 173,200 \left(1 - \frac{86,100 - 5,120}{99,43}\right) = 13,000 + 57,000 = 70,000 \text{ cfs Elav. } 955$$

$$Q_{f27} = 13,000 + 173,200 \left(1 - \frac{86,000 - 5,120}{99,43}\right) = 13,000 + 38,000 = 51,000 \text{ cfs Elav. } 955$$

$$Q_{p27} = 13,000 + 173,200 \left(1 - \frac{86,500 - 5,120}{99,43}\right) = 13,000 + 15,500 = 28,500 \text{ cfs Elav. } 957$$

$$Q_{p2} = 35.3 \text{ cfs; Elav. } = 160.7 \quad \text{From diagram } = 5.5'$$

Dam height = 112 ft. Water surface elevation = 117.3 ft. 117

Water age area above Dam = 45.8 mi. cu.

Normal crest discharge = 38,000 cfs.

Dam failure flow =  $q_{f1} = C_2 T \cdot A_1 \cdot \sqrt{2g}$

$$A_1 = 957.3 \cdot 731 = 226.3'$$

$$T = \frac{1}{2} \cdot 17.3 \cdot 117.3 - 112 = 35.$$

$$q_{f1} = 35 \cdot 226.3 = 370$$

$$A_2 = 45.8 \cdot 370 = 170$$

$$q_{f2} = 35/2 \cdot 170 \cdot \sqrt{2g} = 35,900 \text{ cfs.}$$

Storage in reservoir at elev. 957.3 = 92,810 cu. ft.

DAM FAILURE

NON-UNIFORM FLOW Dam 2, 17

First Reach: 600 ft. M.L. from intake Dam

Length = 14, 20 ft

Storage length = 13.

Velocity  $X = 5.5 \pm 4.1 \text{ ft}$

$$A = 53.3 \beta_3 + 1.2 Y^2; z = 1.5 Y^{1/2} + 5.5 \\ = 100 + 50Y + 1.5Y^2$$

$$S = (732 - 445) / 14,000 = .023$$

$$n = .055 \quad f = (1.45 \cdot 5.5)^{1/2} \cdot 1.5^{1/2} = 4.2030 \text{ A.R. } s$$

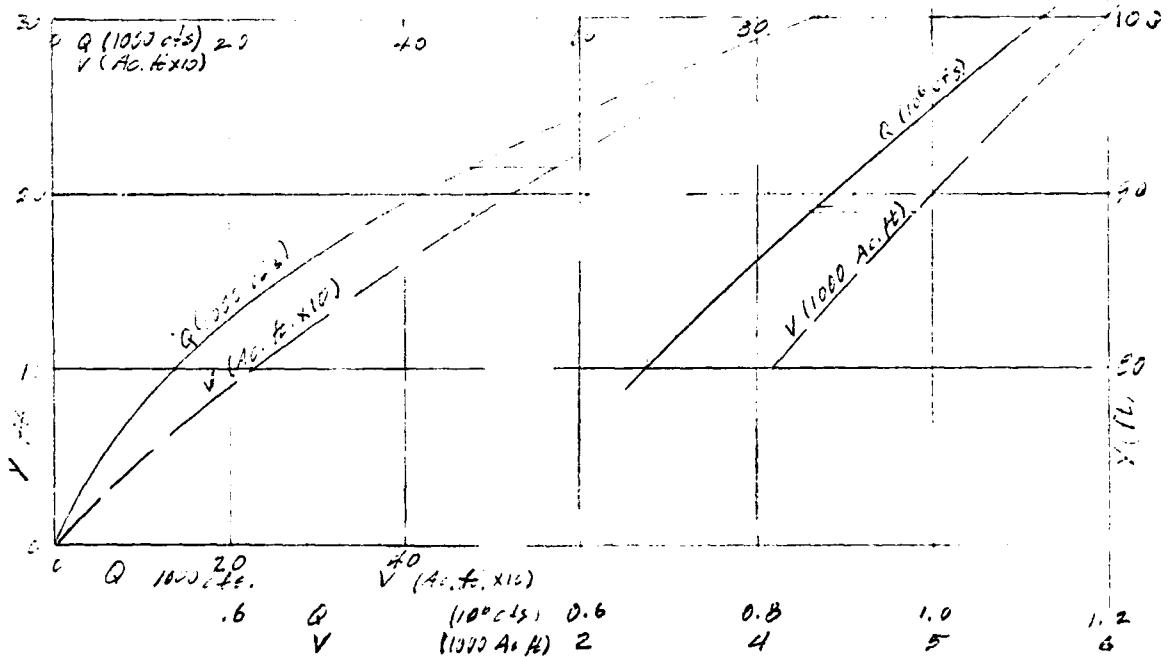
$$Y = 10'; R = 3.530'; V = 15.70 \text{ ft/s}; z = 7.59^{1/2}; \gamma = 224 \text{ lb/cu ft}, \rho = 1.0 \text{ g/cm}^3$$

$$Y = 13'; R = 13.65'; Q = 57,524 \text{ cfs}; A = 2485^{1/2}; V = 3.30 \text{ A.R. s}, \rho = 2554 \text{ g/cm}^3$$

$$Y = 19'; R = 43.77'; Q = 1,127,731 \text{ cfs}, A = 29,100^{1/2}; V = 0.000-16.11.$$

$$Y = 25'; R = 64.50'; Q = 884,840 \text{ cfs}, A = 16,575^{1/2}; V = 6060 \text{ A.R. s}, \rho = 52 \text{ ft}^3$$

$$Y = 30'; R = 40.33'; Q = 673,156 \text{ cfs}, A = 13,700^{1/2}; V = 4520 \text{ A.R. s}, \rho = 29 \text{ ft}^3$$



$$Q_{base} = 38,000 \text{ cfs}; Y = 13.0'; A = 13^{1/2}; V = 473 \text{ A.R. s}, V/1 = 23.8 \text{ ft}^3$$

$$Q_{p1} = 923,700 \text{ cfs}; Y = 30.8'; A = 13,700^{1/2}; V = 5,075 \text{ A.R. s}, V/1 = 53.1 \text{ ft}^3.$$

$$Q_{p2} = 38,000 + 865,900 \left(1 - \frac{5075 - 473}{50,000}\right) = 38,000 + 922,160 = 862,000 \text{ cfs}; Y = 33.7'$$

$$V = 4,890 \text{ A.R. s}, V_{ave} = (5075 + 4890)/2 = 4932.5 \text{ A.R. s}.$$

$$Q_{p3} = 38,000 + 865,900 \left(1 - \frac{4985 - 473}{50,000}\right) = 38,000 + 922,150 = 860,100 \text{ cfs}; Y = 33.7'$$

Detl Spillway

Max. head = 15.7 ft. H.L. = 17  
H = 1.7 ft.

INTAKE Dams

$$Sp. Wt. = \text{specific weight} = 62.4 \text{ lb/ft}^3$$
$$\text{crest elev.} = 15.7 \text{ ft}$$
$$\text{crest elev.} = 15.7 \text{ ft}$$
$$Q = 3.9 (H + h_v)^{3/2} L = 668,100 \text{ cfs}$$

Non overflow section: length = 55 - 40' = 15'

Width may be 20' or 3' wide with runnings.

$$Top elev = 15.7 \text{ ft}$$

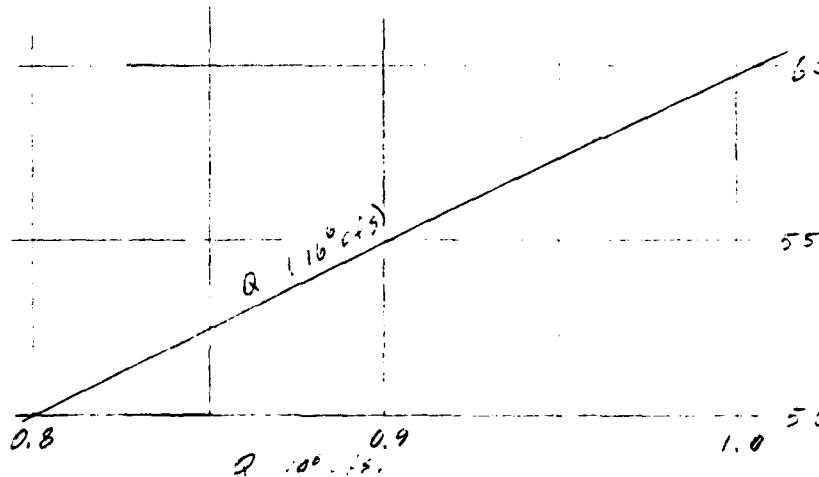
$$Riverbed = 4.1 \text{ ft}$$

$$Q = 3.9 (H + h_v)^{3/2} L = 463,100 \text{ cfs}$$

$$Slope = 1/15 \text{ or } 1/20$$

$$H_v \quad H \quad Q \quad H \quad L \quad Q$$

0.1	8	13,470	-	-	13,470
0.2	15.3	31,740	55	6,150	37,890
Avg. of 50% of 0.1		7.6	2,900	0	
0.2	14	32,430	6	6,480	33,510
3.8	50	500,200	42	300,530	600,730
4.0	55	561,120	47	321,000	732,120
4.2	60	624,300	52	353,000	1,037,300
		57.8			961,000



OAK ENGINEER

1950 COEFFICIENT D.A. 4/17

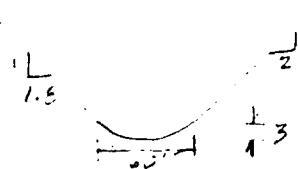
REACH 2 Intake Ditch + minimum flow

Reach length: 16,900'

$$S_{max} = (24.5 - 2.50) / 16,900 = 0.05\%$$

$$M = 0.050$$

Valley x-section:



$$A = 63(3)^{2/3} + 3.3Y^{2/3} + 3.3Y - 2Y^{2/3} = 120 + 6.4Y + 1.3Y^2$$

$$J = (1.446 \cdot 1) S^{1/2} A R^{2/3} = 2.640 A R^{2/3}$$

$$Y = 15; R = 14.233'; Q = 27,050 cfs; A = 14,930^2; V = 562 A.C.H.$$

$$Q = 18,746 cfs.$$

$$Y = 25; R = 14.26'; Q = 44,010 cfs; A = 23,030^2; V = 577 A.C.H.$$

$$Q = 21,452 cfs.$$

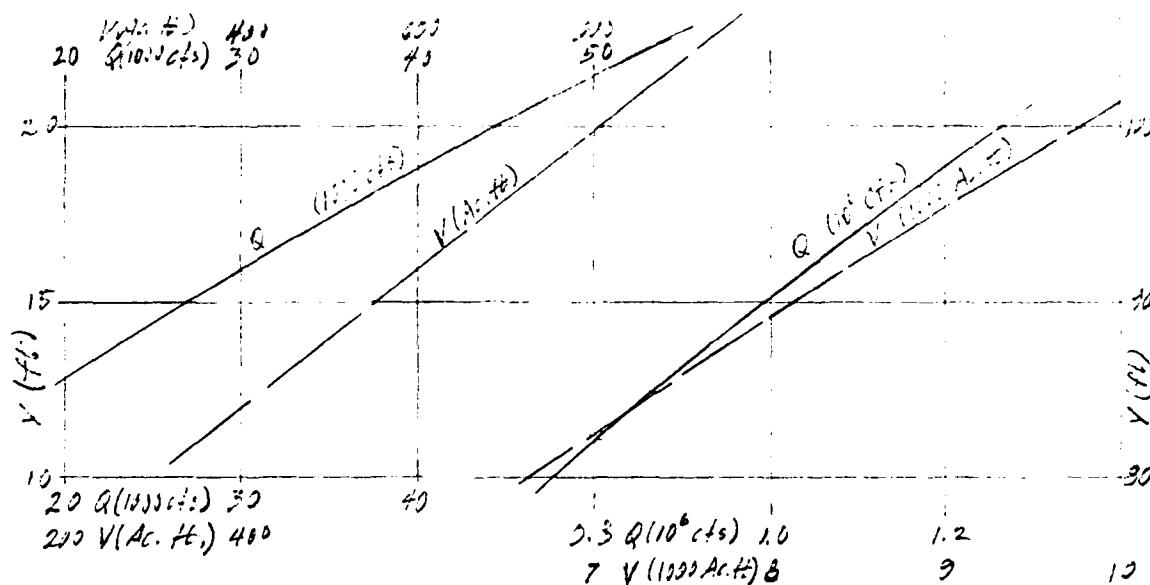
$$Y = 35; R = 14.31'; Q = 72,020 cfs; A = 32,120^2; V = 594 A.C.H.$$

$$Q = 23,575 cfs.$$

$$Y = 45; R = 14.36'; Q = 99,030 cfs; A = 41,220^2; V = 612 A.C.H.$$

$$Q = 24,695 cfs.$$

$$Y = 55; R = 14.41'; Q = 126,040 cfs; A = 50,320^2; V = 630 A.C.H.$$



$$Q_{base} = 38,000 cfs; Y = 18.3'; V = 720 A.C.H.$$

$$Q_p1 = 86,950 cfs; Y = 24.75'; V = 7350 A.C.H.$$

$$Q_{p2T} = 39,000 + 822,950 \left(1 - \frac{7350 - 720}{90,800}\right) = 39,000 + 762,860 = 800,860 cfs; Y = 22.4'$$

$$V = 6990 A.C.H.; V_{ave} = (6990 + 7350)/2 = 7170 A.C.H.$$

$$Q_{p2Z} = 38,000 + 822,950 \left(1 - \frac{7170 - 720}{90,800}\right) = 38,000 + 764,500 = 802,500 cfs; Y = 22.1'$$

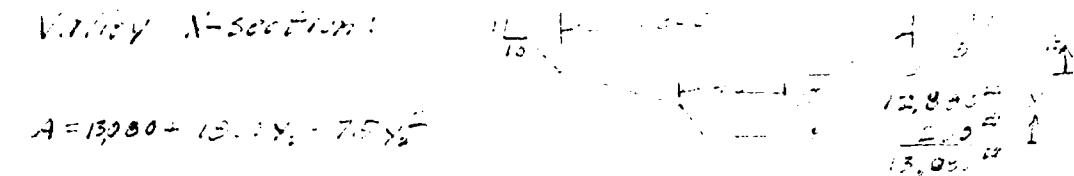
DAM Elevation 5'

REACH 3

Northwest R.R. + 2300' above bottom Bankline  
Length = 3.00'; S = 30'/100' = 0.3 (use 0.3 for bottom line elevation)

$$H = 0.3; Q = 4,600 \text{ cfs}; A = 2,335 \text{ ac. ft.}$$

V.E. 10' N-500 cfs:



$$A = 13,080 + 15.18 \cdot 7.5 \cdot 50$$

$$\begin{aligned} & A = 13,080 + 15.18 \cdot 7.5 \cdot 50 \\ & \quad \frac{1}{2} \cdot 110 \cdot 10 + 15.18 \cdot 7.5 \cdot 50 \\ & \quad 12.83 \text{ ft} \\ & \quad \underline{\underline{13,080}} \text{ ac. ft} \end{aligned}$$

$$Y = 2.5'; Q = 1,100 \text{ cfs}$$

$$Y = 5'; Q = 3,720 \text{ cfs}$$

$$Y = 17'; Q = 5,413 \text{ cfs}$$

$$Y = 15.85'; Q = 38,000 \text{ cfs}$$

$$Y = 40'; Q = 920,450 \text{ cfs}$$

$$Y = 35'; Q = 638,800 \text{ cfs}$$

$$Y = 30'; Q = 416,650 \text{ cfs}$$

$$Y = 30.2'; Q = 802,500 \text{ cfs}$$

$$1. V(1000 \text{ ac. ft}) 2$$

$$2.0 \quad Q(1000 \text{ cfs}) 6.0$$

$$3.0 \quad 120$$

$$15 \quad Y(\text{ft})$$

$$20 \quad V(1000 \text{ ac. ft})$$

$$25 \quad Q(1000 \text{ cfs})$$

$$30 \quad 6.0$$

$$35 \quad V(1000 \text{ ac. ft})$$

$$40 \quad 120$$

$$45 \quad Y(\text{ft})$$

$$50 \quad V(1000 \text{ ac. ft})$$

$$55 \quad 6.0$$

$$60 \quad V(1000 \text{ ac. ft})$$

$$65 \quad 9$$

$$70 \quad 10$$

$$75 \quad Y(\text{ft})$$

$$80 \quad V(1000 \text{ ac. ft})$$

$$85 \quad 120$$

$$90 \quad Y(\text{ft})$$

$$95 \quad V(1000 \text{ ac. ft})$$

$$100 \quad 6.0$$

$$105 \quad V(1000 \text{ ac. ft})$$

$$110 \quad 9$$

$$115 \quad 10$$

$$120 \quad Y(\text{ft})$$

$$125 \quad V(1000 \text{ ac. ft})$$

$$130 \quad 120$$

$$135 \quad Y(\text{ft})$$

$$140 \quad V(1000 \text{ ac. ft})$$

$$145 \quad 6.0$$

$$150 \quad V(1000 \text{ ac. ft})$$

$$155 \quad 9$$

$$160 \quad 10$$

$$165 \quad Y(\text{ft})$$

$$170 \quad V(1000 \text{ ac. ft})$$

$$175 \quad 120$$

$$180 \quad Y(\text{ft})$$

$$185 \quad V(1000 \text{ ac. ft})$$

$$190 \quad 6.0$$

$$195 \quad V(1000 \text{ ac. ft})$$

$$200 \quad 9$$

$$205 \quad 10$$

$$210 \quad Y(\text{ft})$$

$$215 \quad V(1000 \text{ ac. ft})$$

$$220 \quad 120$$

$$225 \quad Y(\text{ft})$$

$$230 \quad V(1000 \text{ ac. ft})$$

$$235 \quad 6.0$$

$$240 \quad V(1000 \text{ ac. ft})$$

$$245 \quad 9$$

$$250 \quad 10$$

$$255 \quad Y(\text{ft})$$

$$260 \quad V(1000 \text{ ac. ft})$$

$$265 \quad 120$$

$$270 \quad Y(\text{ft})$$

$$275 \quad V(1000 \text{ ac. ft})$$

$$280 \quad 6.0$$

$$285 \quad V(1000 \text{ ac. ft})$$

$$290 \quad 9$$

$$295 \quad 10$$

$$300 \quad Y(\text{ft})$$

$$305 \quad V(1000 \text{ ac. ft})$$

$$310 \quad 120$$

$$315 \quad Y(\text{ft})$$

$$320 \quad V(1000 \text{ ac. ft})$$

$$325 \quad 6.0$$

$$330 \quad V(1000 \text{ ac. ft})$$

$$335 \quad 9$$

$$340 \quad 10$$

$$345 \quad Y(\text{ft})$$

$$350 \quad V(1000 \text{ ac. ft})$$

$$355 \quad 120$$

$$360 \quad Y(\text{ft})$$

$$365 \quad V(1000 \text{ ac. ft})$$

$$370 \quad 6.0$$

$$375 \quad V(1000 \text{ ac. ft})$$

$$380 \quad 9$$

$$385 \quad 10$$

$$390 \quad Y(\text{ft})$$

$$395 \quad V(1000 \text{ ac. ft})$$

$$400 \quad 120$$

$$405 \quad Y(\text{ft})$$

$$410 \quad V(1000 \text{ ac. ft})$$

$$415 \quad 6.0$$

$$420 \quad V(1000 \text{ ac. ft})$$

$$425 \quad 9$$

$$430 \quad 10$$

$$435 \quad Y(\text{ft})$$

$$440 \quad V(1000 \text{ ac. ft})$$

$$445 \quad 120$$

$$450 \quad Y(\text{ft})$$

$$455 \quad V(1000 \text{ ac. ft})$$

$$460 \quad 6.0$$

$$465 \quad V(1000 \text{ ac. ft})$$

$$470 \quad 9$$

$$475 \quad 10$$

$$480 \quad Y(\text{ft})$$

$$485 \quad V(1000 \text{ ac. ft})$$

$$490 \quad 120$$

$$495 \quad Y(\text{ft})$$

$$500 \quad V(1000 \text{ ac. ft})$$

$$505 \quad 6.0$$

$$510 \quad V(1000 \text{ ac. ft})$$

$$515 \quad 9$$

$$520 \quad 10$$

$$525 \quad Y(\text{ft})$$

$$530 \quad V(1000 \text{ ac. ft})$$

$$535 \quad 120$$

$$540 \quad Y(\text{ft})$$

$$545 \quad V(1000 \text{ ac. ft})$$

$$550 \quad 6.0$$

$$555 \quad V(1000 \text{ ac. ft})$$

$$560 \quad 9$$

$$565 \quad 10$$

$$570 \quad Y(\text{ft})$$

$$575 \quad V(1000 \text{ ac. ft})$$

$$580 \quad 120$$

$$585 \quad Y(\text{ft})$$

$$590 \quad V(1000 \text{ ac. ft})$$

$$595 \quad 6.0$$

$$600 \quad V(1000 \text{ ac. ft})$$

$$605 \quad 9$$

$$610 \quad 10$$

$$615 \quad Y(\text{ft})$$

$$620 \quad V(1000 \text{ ac. ft})$$

$$625 \quad 120$$

$$630 \quad Y(\text{ft})$$

$$635 \quad V(1000 \text{ ac. ft})$$

$$640 \quad 6.0$$

$$645 \quad V(1000 \text{ ac. ft})$$

$$650 \quad 9$$

$$655 \quad 10$$

$$660 \quad Y(\text{ft})$$

$$665 \quad V(1000 \text{ ac. ft})$$

$$670 \quad 120$$

$$675 \quad Y(\text{ft})$$

$$680 \quad V(1000 \text{ ac. ft})$$

$$685 \quad 6.0$$

$$690 \quad V(1000 \text{ ac. ft})$$

$$695 \quad 9$$

$$700 \quad 10$$

$$705 \quad Y(\text{ft})$$

$$710 \quad V(1000 \text{ ac. ft})$$

$$715 \quad 120$$

$$720 \quad Y(\text{ft})$$

$$725 \quad V(1000 \text{ ac. ft})$$

$$730 \quad 6.0$$

$$735 \quad V(1000 \text{ ac. ft})$$

$$740 \quad 9$$

$$745 \quad 10$$

$$750 \quad Y(\text{ft})$$

$$755 \quad V(1000 \text{ ac. ft})$$

$$760 \quad 120$$

$$765 \quad Y(\text{ft})$$

$$770 \quad V(1000 \text{ ac. ft})$$

$$775 \quad 6.0$$

$$780 \quad V(1000 \text{ ac. ft})$$

$$785 \quad 9$$

$$790 \quad 10$$

$$795 \quad Y(\text{ft})$$

$$800 \quad V(1000 \text{ ac. ft})$$

$$805 \quad 120$$

$$810 \quad Y(\text{ft})$$

$$815 \quad V(1000 \text{ ac. ft})$$

$$820 \quad 6.0$$

$$825 \quad V(1000 \text{ ac. ft})$$

$$830 \quad 9$$

$$835 \quad 10$$

$$840 \quad Y(\text{ft})$$

$$845 \quad V(1000 \text{ ac. ft})$$

$$850 \quad 120$$

$$855 \quad Y(\text{ft})$$

$$860 \quad V(1000 \text{ ac. ft})$$

$$865 \quad 6.0$$

$$870 \quad V(1000 \text{ ac. ft})$$

$$875 \quad 9$$

$$880 \quad 10$$

$$885 \quad Y(\text{ft})$$

$$890 \quad V(1000 \text{ ac. ft})$$

$$895 \quad 120$$

$$900 \quad Y(\text{ft})$$

$$905 \quad V(1000 \text{ ac. ft})$$

$$910 \quad 6.0$$

$$915 \quad V(1000 \text{ ac. ft})$$

$$920 \quad 9$$

$$925 \quad 10$$

# DAM FAILURE

REACH 4 At Hartman's Bridge (Granville Rd.) HAZARD AREA 4

Reach Length = 3500'; S = 30.10,500 = .003; n = 0.535  
 $Q = (1.480/n)^{5/3} A R^{2/3} = 2.3255 A R^{2/3}$

Dam: L = 140'; C = 3.4; G = 3.7L(H + V)^2 =

H = 12; H\_V = 16; Q\_f = 22,310 cfs. over dam  
 Y = 4'; V\_d = 9145; Q\_f =  $\frac{3,360,000}{15,670,000}$ ; A = 430 ft^2  
 $G_f = 25,670 \text{ cfs}, A_f = 2160^2, V = 174,400 \text{ cu. ft.}$

H = 15'; H\_V = 10'; Q\_f = 35,500 cfs. over dam  
 Y = 7'; V\_d = 9145; Q\_f =  $\frac{7,360,000}{20,950,000}$ ; A = 700 + 245 + 130 + 65 = 1140 ft^2  
 $G_f = 20,950 \text{ cfs}, V = (193 + 211) 3500 / 43,500 = 235 \text{ cu. ft.}$

H = 17'; H\_V = 10'; Q\_f = 27,650 cfs.  
 Y = 6'; V\_d = 9145; Q\_f =  $\frac{7,440,000}{35,090,000}$ ; A = 600 + 190 + 120 + 50 = 950 ft^2  
 $G_f = 35,090 \text{ cfs}, V = (930 + 1960) 3500 / 43,500 = 237 \text{ cu. ft.}$

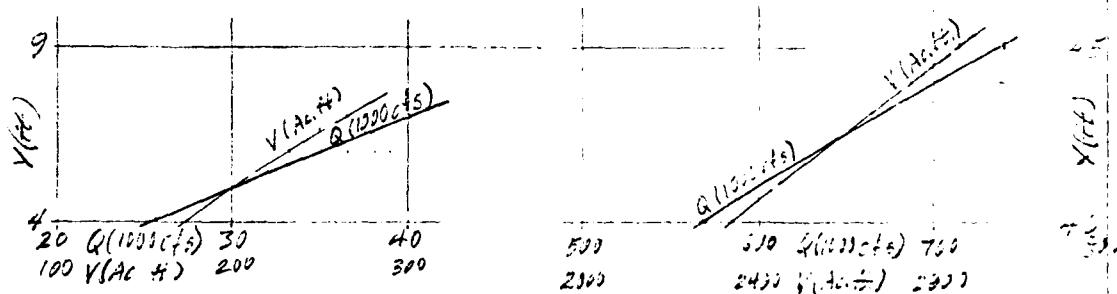
For greater flows the dam will be submerged by the water.  
 $Q = 2.3255 A R^{2/3}; A = 1630 + 400 \times 3; + 300 \times \frac{Y}{2} + 25 \times \frac{Y}{2}^2; Y_1 = Y - 4.5$

$Y = 44; Q = 731,900 \text{ cfs}; A_f = 34,160^2; V = 2745 \text{ cu. ft.}$

$Y = 45; Q = 731,900 \text{ cfs}; A_f = 35,470^2; V = 2,933 \text{ cu. ft.}$

$Y = 45.5; Q = 731,900 \text{ cfs}; A_f = 35,500^2; V = 2,930 \text{ cu. ft.}$

$Y = 46; Q = 569,100 \text{ cfs}; A_f = 29,160^2; V = 2,340 \text{ cu. ft.}$



$Q = 38,000 \text{ cfs}; V = 251 \text{ cu. ft.}; Y = 6.6'; \text{Elev.} = 197'; \text{Area} = 320 \times 5.25^2 / 2 = 172 \text{ ac.}$

$$Q_{p2T} = 38,000 + 701,500 \left(1 - \frac{2,900 - 251}{70,500}\right) = 38,290 + 691,300 = 713,390 \text{ cfs}, Y = 44.5$$

$$V = 2800 \text{ cu. ft.}, V_{30} = (2900 + 2300) \times 2 = 28,300$$

$$Q_{p2} = 3900 + 701,500 \left(1 - \frac{2,970 - 251}{90,500}\right) = 35,200 + 631,600 = 710,350 \text{ cfs}, Y = 44.5$$

$$\text{Elev.} = 237'; \text{Area} = 4500 \times 5500 / 2 = \frac{24,750}{2200 \times 2000} = \frac{135}{340} \text{ ac.}$$

Depth over top = 237' - 160.10 = 76.87' above my top =  $\frac{1}{3} - \frac{1}{4}$

D. 60' : 1000 ft. long reach from 202 to 212 MSL.  
 Reach 5 : 1000 ft. long reach from 182 to 192 MSL.  
 Area of valley =  $1000 \times 1000 = 1,000,000$  ft<sup>2</sup>  
 Area of 200 ft. =  $200 \times 1000 = 2,000,000$  ft<sup>2</sup>  
 Ratio of change =  $1000/200 = 5$  times.

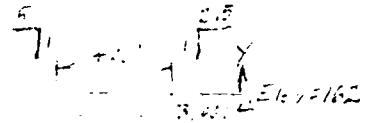
REACH 6 : 1000' long from 212 to 22.2 MSL

$$\text{Length} = 1000'; S = (182 - 162)/1000 = 0.020; n = 0.035$$

VALLEY X-SECTION

$$Q = (1.45/n)S^{1/2}, AR^{2/3} = 2,439, A \bar{y}^{2/3}$$

$$A = 300x + 400y + 3,750^2$$



$$\text{D.A.M.} : L = 160'; C = 3.5$$

$$R = 3.9(160)(H + H_0)^{3/2}$$

$$H_0 = (35,000/3.5(160))^{2/3} = 11.6$$

$$= 14.56 - 3.5' = 11.0'$$

$$V = 7554(160/\sqrt{11.0}) + (153 - 7554)11.0 = 271,540 \text{ cu. ft.}$$

$$H = 23'; Q = 77,720 \text{ cfs.}$$

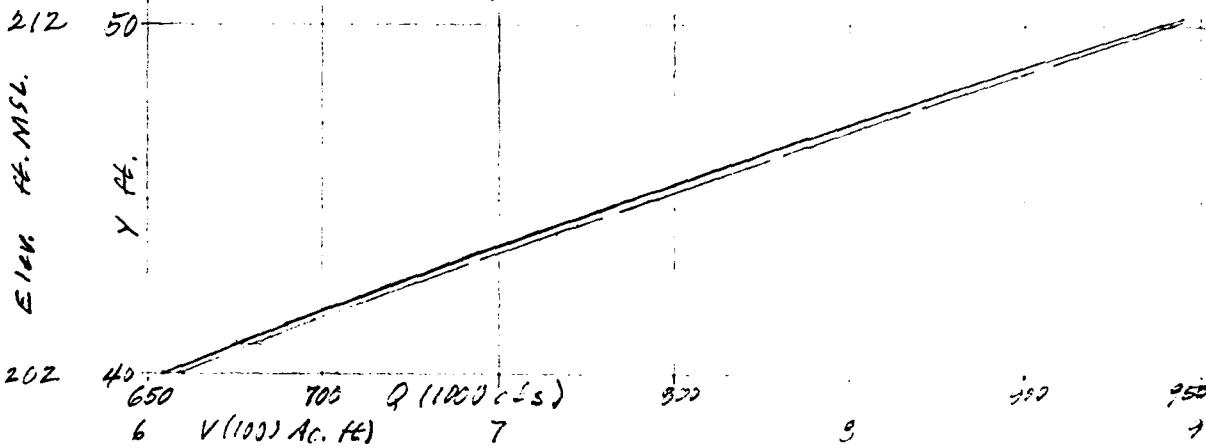
For protection of towns the dam will be increased by 3.5'.

Using Valley X-Section :  $Q = 2,439, A \bar{y}^{2/3}$

$$Y = 40'; Q = 654,050 \text{ cfs}; A = 25,000^2; V = 56,340,000 + 262(40,000) = 6102,600$$

$$Y = 50'; Q = 938,000 \text{ cfs}; A = 32,375^2; V = 1115,400,000 + 312(50,000) = 9915,400,000$$

$$Y = 44.0'; Q = 760,300 \text{ cfs}; A = 27,530^2; V = 955,400,000 + 252(44,000) = 7163,400,000$$



$$Q_p = 719,600 \text{ cfs}; Y = 42.9' = 204.9 \text{ ft.}; V = 6750 \text{ cu. ft.}$$

$$Q_{1,2} = 38,000 + 681,600 \left(1 - \frac{6750 - 919}{90,800}\right) = 38,000 + 637,910 = 675,910 \text{ cfs.}$$

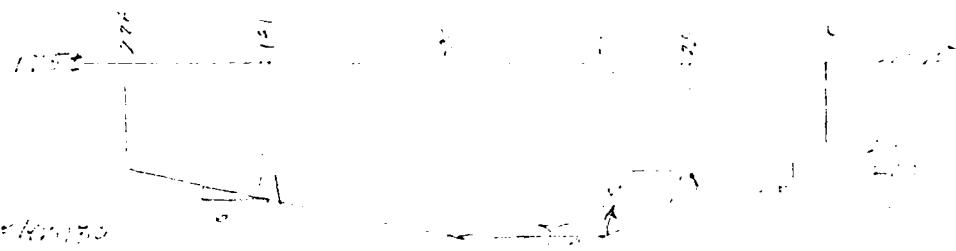
$$V = 6300 \text{ cu. ft.}; V_{1,2} = (6300 + 6750)\Delta t = 6725,400 \text{ ft.}; Y = 45.7'$$

$$Q_{p,2} = 38,000 + 681,600 \left(1 - \frac{6525 - 919}{90,000}\right) = 38,000 + 639,510 = 677,510 \text{ cfs.}$$

$$\text{Area} = \frac{1200' \times 1000'}{2740} = \frac{2,400,000}{2740} = 877.50 \text{ MSL}$$

Open Channel

Louisville, KY - Kentucky River - 100' wide - 10' deep - 1000 cfs



$$Y = 10 \text{ ft} \quad S = 0.020 \quad F_0 = 5.0$$

$$A = 1000 \text{ cfs} \quad R = 12.5^{\frac{1}{2}} \quad D = 1000 \text{ cfs}$$

$$155' \quad Y = 25'; Q = 57,700 \text{ cfs}$$

$$155' \quad Y = 30'; Q = 46,700 \text{ cfs}$$

$$155' \quad Y = 35'; Q = 35,300 \text{ cfs}$$

$$155' \quad Y = 35'; Q = 123,700 \text{ cfs}$$

$$145' \quad Y = 15'; Q = 19,400 \text{ cfs}$$

$$145' \quad Y = 10'; Q = 9,000 \text{ cfs}$$

$$Y = 4.7; Q = 1000 \text{ cfs}$$

$$Q = 5,375 \text{ cfs}$$

$$A = 3,000 \text{ ft}^2$$

$$T = 5.00 \text{ ft}$$

$$A = 1,920 \text{ ft}^2$$

$$A = 6,075 \text{ ft}^2$$

$$Y = 10.7; Q = 1000 \text{ cfs}$$

$$Q = 14,700 \text{ cfs}$$

$$T = 1.3 \text{ ft}$$

$$D = 16.1 \text{ ft}$$

For major overbank restriction:  $Q = A \sqrt{2g} (H_v + Kx)$

$$K = 0.00 \quad \text{overbank} \quad x = 1.2 \quad H_v = 5.0 \quad S = 0.2'$$

$$155' \quad Y = 20'; A = 7,520 \text{ ft}^2; H_v = 1.0'; H_T = 7.0'; Q = 57,700 \text{ cfs}$$

$$160' \quad Y = 30'; A = 5,200 \text{ ft}^2; H_v = 1.8'; H_T = 8.8'; x = 1.9; S = 0.2'$$

$$165' \quad Y = 35'; A = 3,700 \text{ ft}^2; H_v = 2.0'; H_T = 9.0'; x = 1.9; S = 0.2'$$

$$170' \quad Y = 40'; A = 2,520 \text{ ft}^2; H_v = 2.2'; H_T = 9.5'; x = 1.9; S = 0.2'$$

$$175' \quad Y = 45'; A = 1,800 \text{ ft}^2; H_v = 2.3'; H_T = 10.0'; x = 2.0; S = 0.2'$$

$$180' \quad Y = 25'; A = 4,000 \text{ ft}^2; H_v = 1.6'; H_T = 7.0'; Q = 97,500 \text{ cfs} \quad S = 0.2'$$

South Meadow Road Bridge - 25' overpass; also Silver St.

$$Q = A \sqrt{2g} (H_v + Kx) \quad H_v = 2.5 \quad K = 1.2$$

$$155' \quad Y = 16'; A = 1,700 \text{ ft}^2; H_v = 1.5'; H_T = 15.1 \text{ ft}$$

$$155' \quad Y = 11'; A = 2,700 \text{ ft}^2; H_v = 1.5'; H_T = 5.6 \text{ ft}$$

$$155' \quad Y = 21'; A = 11,700 \text{ ft}^2; H_v = 1.5'; H_T = 21.1 \text{ ft}$$

$$170' \quad Y = 6'; A = 300 \text{ ft}^2; H = 2'; Q = 3,540 \text{ cfs}$$

$$100' \quad Y = 16'; A = 800 \text{ ft}^2; H = 4'; Q = 12,570 \text{ cfs}$$

$$170' \quad Y = 26'; A = 1300 \text{ ft}^2; H = 7'; Q = 26,250 \text{ cfs}$$

$$175' \quad Y = 31'; A = 1530 \text{ ft}^2; H = 8'; Q = 33,300 \text{ cfs}$$

$$\text{Thru city: } n = 0.040; s = .001; Q = 1,174,840 \text{ cfs}$$

← North Meadow Road  
County St. 25' overpass

$$155' \text{ ASL } Y = 10'; Q = 46,700 \text{ cfs}; A = 13,600 \text{ ft}^2; H_v = 15.5 \text{ ft}$$

$$165' \quad Y = 15'; Q = 146,600 \text{ cfs}; A = 27,375 \text{ ft}^2; H_v = 241,000 \text{ cfs}$$

$$165' \quad Y = 20'; Q = 293,200 \text{ cfs}; A = 41,700 \text{ ft}^2; H_v = 462,000 \text{ cfs}$$

$$175' \quad Y = 30'; Q = 631,900 \text{ cfs}; A = 72,000 \text{ ft}^2; H_v = 912,500 \text{ cfs}$$

$$175' \quad Y = 5'; Q = 7,350 \text{ cfs}; A = 3,400 \text{ ft}^2; H_v = 46,300 \text{ cfs}$$

$$170' \quad Y = 25'; Q = 467,400 \text{ cfs}; A = 56,575 \text{ ft}^2; H_v = 700,375 \text{ cfs}$$

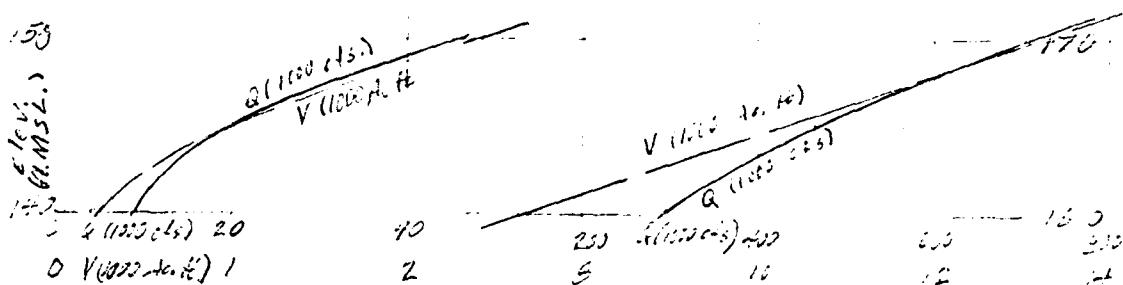
DAM FAILURE

Recon T: Fork - 1000 ft. dam

1000 ft. dam at 1000 ft. elev.

Flow at trough volume

$$\begin{aligned}
 \text{Area} &= 140 \times 100 = 14,000 \text{ ft}^2 \\
 A &= 80 \times \frac{100}{43,560} = 1.84 \text{ ac.} \quad V = \frac{1.84}{4.37} = 0.42 \text{ ft.} \\
 &= 80 \times \frac{100}{43,560} = 1.84 \text{ ac.} \quad V = 0.42 \times 100 = 42 \text{ ft.} \\
 A &= 1.84 \text{ ac.} \quad V = (42 + 5.0) = 47 \text{ ft.} \\
 V_{140} &= 76(1.84) = 138.4 \text{ cu. ft.} \quad V = 47 \times 100 = 4,700 \text{ cu. ft.} \\
 V_{150} &= 76 + 4.0 = 80 \text{ ft.} \quad V_{160} = 7.363.46 \text{ cu. ft.} \quad V_{170} = 1.7343.46 \text{ cu. ft.} \\
 &= 2413.46 \text{ cu. ft.}
 \end{aligned}$$



$$Q = 30,000 \text{ cfs}; V = 2,950 \text{ Ac.H.}; E_{100} = 143.6 \text{ ft. MSL}$$

$$Q_{p1} = 627,500 \text{ cfs}; V = 12,800 \text{ Ac.H.}; E_{100} = 169.4 \text{ ft. MSL}$$

$$Q_{p2} = 39,000 + 639,500 \left(1 - \frac{12,375 - 2,050}{90,800}\right) = 39,000 + 563,900 = 602,900 \text{ cfs}; E_{100} = 167.9 \text{ ft. MSL}$$

$$V = 11,350 \text{ Ac.H.}; V_{200} = (11,350 - 12,800)/2 = 12,375 \text{ Ac.H.}$$

$$Q_{p2} = 39,000 + 639,500 \left(1 - \frac{12,375 - 3050}{90,900}\right) = 39,000 + 566,800 = 594,900 \text{ cfs}; E_{100} = 168.0 \text{ ft. MSL}$$

Stage at R#202:

$$\text{Base flow stage: } S = 0.0020 \quad E_{100} = 0.0020(7600) = 163.2 \text{ ft. MSL}$$

Failure stage:  $S = 0.012$ ;  $E_{100} = 163.04, 02(7600) = 175.6 \text{ ft. MSL}$   
due to large area & distance from failure point

Depth over R#202 NE of bridge:  $169.6 - 164.2 = 5.4 \text{ ft. MSL}$   
 $170 - 164.2 = 5.8 \text{ ft. MSL}$

Depth over main R.R. Trestle:  $169.6 - 164.2 = 5.4 \text{ ft. MSL}$   
 $165.0 - 164.2 = 0.8 \text{ ft. MSL}$

0.417 FALLING

Recon of R.C. Embankment Bridge for downstream slope = vertical ratio 1.05

$$A = \text{Area} = 0.417 \times 100 \times 100 = 41700 \text{ sq. ft.}$$

$$Q = 2417 \text{ cfs} \quad Q_1 = 12,085 \text{ cfs}$$

$$Slope X = 500 \text{ ft/m}$$

$$A_1 = 600 \text{ ft}^2 + 1.05 \times 100 \times 100 = 16500 \text{ ft}^2$$

$$A_2 = 2000 + 2.05X + 2.25Y_1$$

$$A_3 = 0.31 + 4.025Y_2 + 2.25Y_2^2$$

$$R = (1.417/3) S^{1/2} A^{2/3} = 0.417 A S^{2/3}$$

$$Y_1 = 20'; \quad Q = 12,085 \text{ cfs}; \quad A = 4,200 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

$$Y_1 = 15'; \quad Q = 73,400 \text{ cfs}; \quad A = 9,130 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

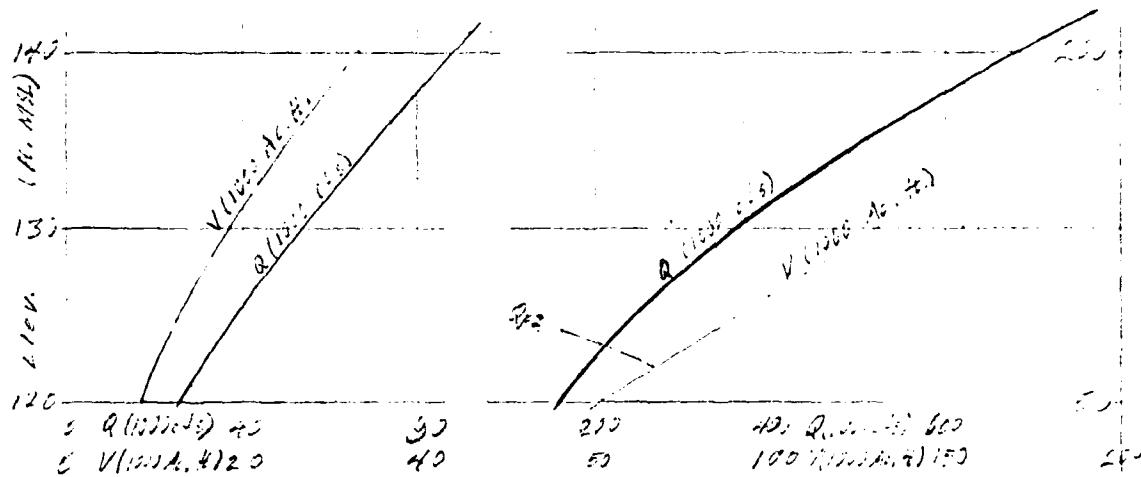
$$Y_2 = 5'; \quad Q = 91,500 \text{ cfs}; \quad A = 11,350 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

$$Y_2 = 40'; \quad Q = 373,500 \text{ cfs}; \quad A = 32,750 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

$$Y_2 = 60'; \quad Q = 321,500 \text{ cfs}; \quad A = 43,150 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

$$Y_2 = 70'; \quad Q = 762,500 \text{ cfs}; \quad A = 50,430 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$

$$Y_2 = 20'; \quad Q = 189,800 \text{ cfs}; \quad A = 13,300 \text{ ft}^2; \quad S = 1.21; \quad E = 12.326; \quad Z_1 = 126$$



$$Q = 39,000 \text{ cfs}; \quad V = 12,500 \text{ A.C. ft}; \quad E.L = 124.3 \text{ MSL}$$

$$Q_{p1} = 604,800 \text{ cfs}; \quad V = 192,500 \text{ A.C. ft} > 92,800 \text{ in developing stage - no surcharge}$$

$$Elev: 155; V = 672,000 \text{ A.C. ft}; Q = 192,500 \text{ cfs}; Q_{p1} = 604,800 \left(1 - \frac{672,000 - 12,500}{92,800}\right) = 256,511 \text{ cfs}$$

$$Elev: 160; V = 83,300 \text{ A.C. ft}; Q = 238,500 \text{ cfs}; Q_{p2} = 238,500 + 566,900 \left(1 - \frac{83,300 - 12,500}{70,850}\right) = 162,931 \text{ cfs}$$

$$Elev: 165; V = 77,000 \text{ A.C. ft}; Q = 212,300 \text{ cfs}; Q_{p3} = 58,000 + 566,900 \left(1 - \frac{77,000 - 12,500}{70,850}\right) = 202,200 \text{ cfs}$$

$$Elev: 170.5; V = 76,100 \text{ A.C. ft}; Q = 212,300 \text{ cfs}; Q_{p4} = 58,000 + 566,900 \left(1 - \frac{76,100 - 12,500}{70,850}\right) = 202,490 \text{ cfs}$$

$$\therefore E.L = 157.4; \quad R = 209,000 \text{ cfs}$$

# DAM FAILURE

The COTCHETAT DAM 14.7  
ft. h. & went through during 1972 season as a reservoir  
with upstream - outlet control at Westfield. No spring runoff  
TOWN TIME.

Check time constraints:

$$\text{Durational stage: } T = 125/\rho g A_f = 125/(62.4 \times 373,500/30,000) = 336,100$$

$$T = 52,047 \text{ sec} = 14.4 \text{ hrs}$$

$$\text{Time to crest from first runoff} = 2,361$$

$$t = (94.0 + 16.46) \sqrt{373,500/30,000} = 25,000/12.44 = 2,075.00$$

$$t = 35 \text{ min.}$$

$$T/t = 14.6,600/35 = 25.3 \text{ H to maximum static headwater}$$

Stage Velocity:

$$\text{Elev. 100} \quad A = 0$$

$$\text{Elev. 150} \quad A = 3100 \text{ ac.} \quad V = A/t = 51,750 \text{ ft/sec.}$$

$$\text{Elev. 200} \quad A = 6230 \text{ ac.} \quad V = A/t = 23,150 \text{ ft/sec.}$$

$$\text{check } \Delta V = (A_1 + A_2)/2 = 50(3100 + 6230)/2 = 23,450 \text{ ft/sec.}$$

$$\text{Elev. 150} \quad Q_{P2T} = 308,600 + 570,700 \left(1 - \frac{145,000 - 93,000}{30,000}\right) = 333,300 - 237,600 = 54,200 \text{ cfs.}$$

$$\text{Elev. 191} \quad Q_{P2T} = 309,600 + 570,700 \left(1 - \frac{166,500 - 93,000}{30,000}\right) = 339,600 + 109,700 = 449,300 \text{ cfs.}$$

$$\text{Elev. 137.5} \quad Q_{P2} = 510,000 \text{ cfs} = 303,600 + 206,400$$

Westfield River & Little River proposed dam at 14.7 ft.  
Little River at RR bridge proposed dam at 12.5 ft.

Dam Estimating

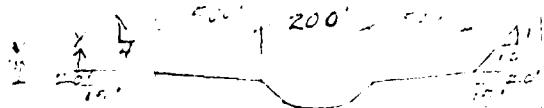
Reach & Meeting - most springtail 2000 ft. 1000 ft. 1000 ft.

Max. Class. Min. Est. 12,11

$$\text{Elevation } 1000 \text{ ft} = 17,260 \text{ ac. ft. } 25.1 \text{ ft. } 12 \text{ ft. } n = 0.8$$

$$A = (1000 \times 5) 5^2 AR^{0.5} = 1.257 AR^{0.5}$$

Valley X - 500 ft. m:



$$A = 200 \times 30 + 250(2) + 250(100)$$

$$+ 250Y^2 + 1250Y + 100Y^2 = 10000 + 1250Y + 7Y^2$$

$$Y = 25; A = 115,750 \text{ ac. ft. } V = 16,275 \text{ ac. ft. } V = 7,500 \text{ cu. ft.}$$

$$Y = 10; A = 250,600 \text{ ac. ft. } A = 23,700 \text{ ac. ft. } V = 12,350 \text{ cu. ft.}$$

$$Y = 15; A = 343,400 \text{ ac. ft. } A = 35,575 \text{ ac. ft. } V = 15,000 \text{ cu. ft.}$$

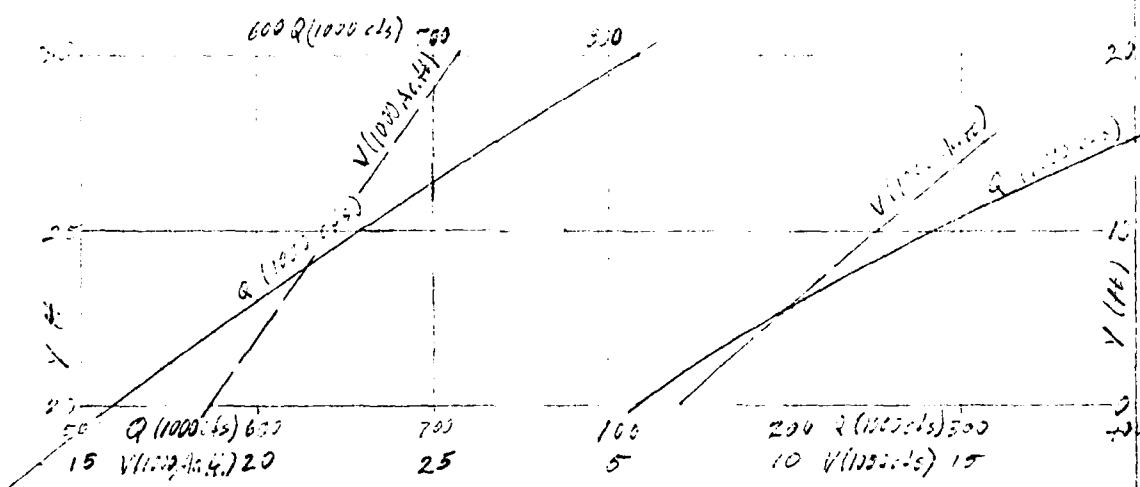
$$Y = 11.2; A = 308,600 \text{ ac. ft. } A = 30,275 \text{ ac. ft. } V = 13,340 \text{ cu. ft.}$$

$$Y = 30; A = 516,100 \text{ ac. ft. } A = 58,750 \text{ ac. ft. } V = 20,000 \text{ cu. ft.}$$

$$Y = 25; A = 450,000 \text{ ac. ft. } A = 50,375 \text{ ac. ft. } V = 18,000 \text{ cu. ft.}$$

$$Y = 20; A = 375,000 \text{ ac. ft. } A = 47,500 \text{ ac. ft. } V = 15,000 \text{ cu. ft.}$$

$$Y = 17.2; A = 33,100 \text{ ac. ft. } A = 6,335 \text{ ac. ft. } V = 2,430,000 \text{ cu. ft.}$$



$$Q_{p2T} = \frac{328,600 + 201,400(1 - \frac{18,450 - 13,340}{90,800})}{90,800} = 308,600 + 130,000 = 438,600 \text{ ac. ft.}$$

$$V = 18,100 \text{ ac. ft.}; V_{avg} = (18,100 + 13,400)/2 = 15,750 \text{ ac. ft.}$$

$$Q_{p2T} = 308,600 + 201,400(1 - \frac{18,275 - 13,340}{90,800}) = 303,600 + 130,500 = 433,100 \text{ ac. ft.}; V = 12,350 \text{ cu. ft.}$$

$$Q_{p1} = 209,000 \text{ ac. ft.}; V = 10,100 \text{ ac. ft.}$$

$$Q_{p2T} = 38,000 + 171,000(1 - \frac{10,100 - 2,930}{70,500}) = 38,000 + 157,500 = 195,500 \text{ ac. ft.}; V = 5,300 \text{ cu. ft.}$$

$$V = 7,300 \text{ ac. ft.}; V_{avg} = (7,300 + 10,100)/2 = 8,700 \text{ ac. ft.}$$

$$Q_{p2T} = 38,000 + 171,000(1 - \frac{9,950 - 2,930}{70,500}) = 38,000 + 157,500 = 195,500 \text{ ac. ft.}; V = 5,300 \text{ cu. ft.}$$

DAM FAILURE

Report of the Committee on Dam Safety

The 1960 Disaster, April 13, 1967

Report of the Committee on Dam Safety

Hydroelectric Power Generation - 1967

$$A = 1,000 \text{ ac}^2; Q = 100 \text{ cfs} = 2,000 \text{ cu ft sec}$$

$$Y_1 = 1000 \text{ ft. elevation}$$

$$A = 1,000 \text{ ac}^2; Q = 100 \text{ cfs} = 2,000 \text{ cu ft sec}$$

$$V = 2 \sqrt{g} Y + \frac{Q}{2} Y^2 = 2(32.2)Y + \frac{2000}{2}(Y)^2 \quad \text{Assume power line east of dam.}$$

$$Y = 50; Q = 53.00 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 50; Q = 567.50 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 50; Q = 513.50 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

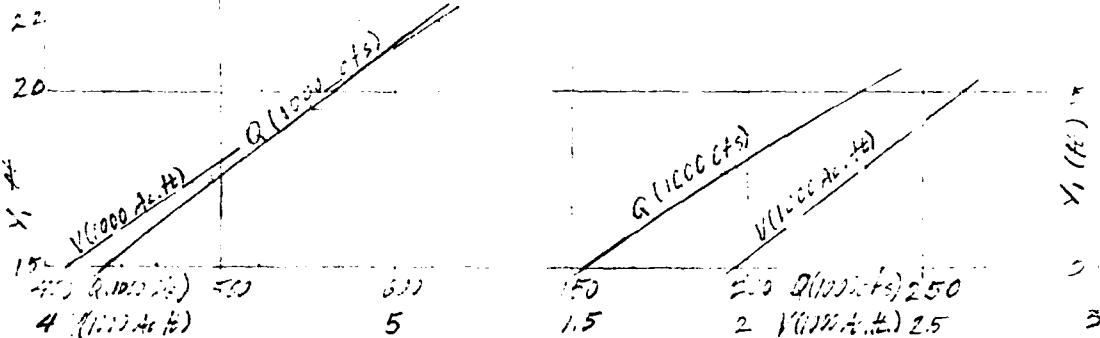
$$Y = 50; Q = 153,400 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 100; Q = 32,600 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 5; Q = 231,740 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 10; Q = 432,500 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$

$$Y = 2.5; Q = 193,400 \text{ cfs}; A = 1,000 \text{ ac}^2; V = 2,330 \text{ ft. elevation}$$



$$Q_{p2} = 38,000 + 158,000 \left(1 - \frac{2,330 - 950}{90,800}\right) = 38,000 + 155,800 = 193,800 \text{ cfs. } t = 2.7 \text{ hr.}$$

$$V = 2310 \text{ ac-ft. } V_{avg} = (2310 + 2330)/2 = 2320 \text{ ac-ft.}$$

$$Q_{p2} = 38,000 + 158,000 \left(1 - \frac{2,320 - 950}{90,800}\right) = 38,000 + 155,800 = 193,800 \text{ cfs.}$$

Peak M.S. Elev. at pyramid on Park Ave. = 23 + 3 = 26 ft. MSL

Deck of Bridge st. at 22 = 23 ft. MSL.

Peak M.S. Elev. at First St. = 25 + 23 + 3 = 51 ft. MSL.

Top of dike at Park Ave. = 26.2 ft. MSL

Top of conc. wall at Park Ave. = 26.2 ft. MSL

DAE 5-22-22

1922 NOV 12 D-25

Dec 1st Memorial Drive Bridge - 100' span - 10' width

Water Variation

$$A = 0.711 + 10.12 \cdot 4.21^{2/3}$$

$$S = 1.75 \cdot 3.022 = .3022$$

$$n = 0.25$$

$$Q = (1.420, 5) S = 1.420 \cdot 0.3022 = 2.755 \text{ ft}^3/\text{s}$$

$$Y = 11.43 \quad q = 30,000 \text{ cfs} \quad Z = 5.750 \text{ ft}$$

$$Y = 31.37 \quad q = 173,000 \text{ cfs} \quad Z = 5.250 \text{ ft}$$

$$Z = 52.9$$

$$Z = 72.1$$

Flood stage elevation 11.43

Probable height of Eastern States Flood

$$6000' \times 30.000 = 200,000 \text{ cfs}$$

Memorial Drive road surface elevation above sea level  
at River st. = 61.3'

DAM FAILURE

1.00 CFS = 1 FT. April 15-17

AT SPRINGFIELD ON CONNECTICUT RIVER

FAVORS THE RIVER

Estimated flow 1 ft.

Current 1 ft. from 1 ft. of river discharge: 33,000 cfs

Flow in New Henniker River at Henniker Lake:  $\frac{16,600}{54,000 \text{ cfs}}$

After 1 ft. rise of 1 ft. in main river

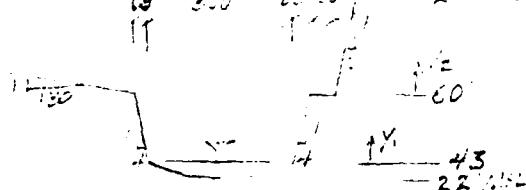
Total flow: 213,000 cfs

Hydrostatic pressure at Springfield along the river line

Valley cross-section (TVA Bridge Design Tab.)

$$S = 1/1000 \text{ in. } f = 0.010$$

$$A_1 = 800(21)^{3/4} + 3.0y + 2/2 + y^2 \\ = 12,630 + 300y + 4y^2$$



$$A_2 = 12,630 + 13,600 + 1156 + 1392 \frac{y^2}{2} + 932 \frac{y}{2} + \frac{2}{2} y^2 \\ = 27,356 + 966 \frac{y}{2} + 66 y^2$$

H.P. =

$$Q = (1.486/m) S^{1/2} A R^{2/3} = 2.223 A R^{2/3}$$

$$Y_1 = 20' \quad Q = 270,400 \text{ cfs}$$

$$\text{Record: } Q = 283,000 \text{ cfs } Y_1 = 20.6'$$

$$Y_1 = 17' \quad Q = 240,200 \text{ cfs}$$

$$Y_1 = 10' \quad Q = 161,200 \text{ cfs}$$

$$Y_1 = 4.5' \quad Q = 210,100 \text{ cfs}$$

$$A = 25,313 \frac{m}{s}$$

ELEV. = 55.5'

$$Y_1 = 5' \quad Q = 113,500 \text{ cfs}$$

$$A = 16,700 \frac{m}{s}$$

$$Y_1 = 1' \quad Q = 80,900 \text{ cfs}$$

$$A = 13,400 \frac{m}{s}$$

$$Y_1 = 0' \quad Q = 73,500 \text{ cfs}$$

$$A = 12,600 \frac{m}{s}$$

This indicates that hydrostatic pressure is less than 1 ft. at 1000 ft. above.

The flow of 54,200 cfs is not a significant flow.

Dam Elevation  
At Springfield for experimental work

Flow = 7,000 cfs

Before failure 1000'

Cable str. Dam capacity total discharge = 33,000 cfs  
Ave. min. connection & net storage = 10,000 cfs  
Total flow = 54,000 cfs

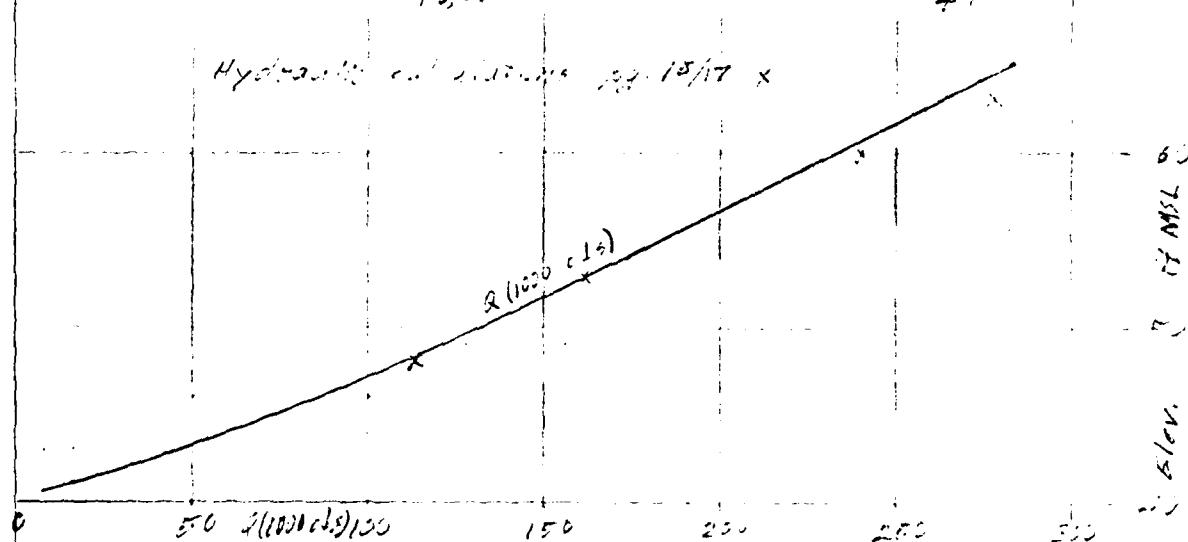
After failure 1000' unconnected:

Total flow = ~~155,000~~  
~~210,000 cfs~~

Connection 1000' cable unbroken - discharge same as before.  
Surface Water stages and Water Flow in cfs.

Q	Stage	Flow
232,000	10.6	6,020 cfs
174,000	10.73	6.5 from parabola
156,000	10.81	5.4
102,000	7.04	
16,000		

Hydrograph at discharge 210,000 cfs



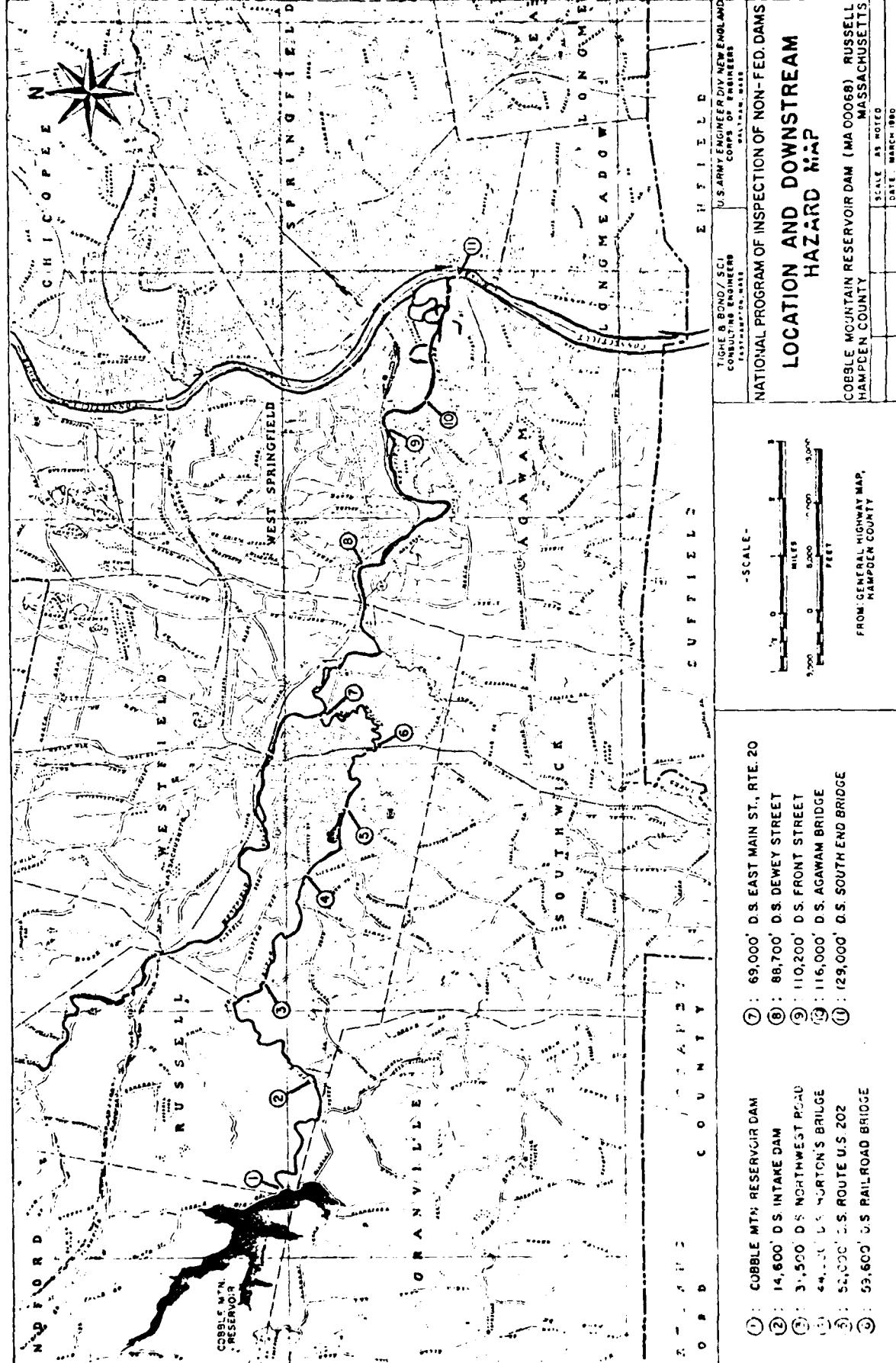
$$Q = 54,000 \text{ cfs} \quad \Sigma h_0 = 43.7$$

$$Q = 210,000 \text{ cfs} \quad \Sigma h_0 = 57.7$$

DAM FAILURES

Map No.	Dist. from Dam	Before dam failure Elev. ft.	After dam failure Elev. ft.	Comments	
				Depth ft.	Depth ft.
1	0 Dam	35,719 19	—	96,400 91	—
2	W.C. Intake dim.	35,652 19	—	96,400 90	Water station flooded by dam failure
3	31,500 Air Intake Stand	35,040 16	6	250 ft. 250 ft.	Water station flooded by dam failure
4	14,500 Air Intake Stand	35,040 16	3	5 180 ft. 180 ft.	110 ft. ac.
5	5,200 Route 200 Bridge	35,199 11	—	67,500 102 ft.	Water station flooded by dam failure
6	3,000 E. Main Bridge	Elev. 164 35,191 11	5	60,500 Elev. 168'	60 ft. ac. 25 ft. ac.
7	2,600 E. Main St.	35,191 Elev. 125	3	80,000 Elev. 137'	35 ft. ac. Water flooded in the area
8	5,700 Dewey St.	35,080 20	3	196,000 35	10 ft. ac.
9	110,200 First St.	35,000 14	—	196,000 26	—
10	116,100 Apavon Rd.	35,050 11	—	194,000 21	210 ft. ac. This is probably the first bridge to fail due to dam failure
11	124,100 South Rd. Bridge	35,050 11	—	194,000 22	—
				210 ft. ac.	100 ft. ac. This is probably the last bridge to fail due to dam failure

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APPENDIX E

INFORMATION AS CONTAINED IN THE  
NATIONAL INVENTORY OF DAMS

# INVENTORY OF DAMS IN THE UNITED STATES

(1)	CITY, COUNTY OR DIVISION	STATE, COUNTY, DIST.	COUNTRY	NAME	LATITUDE NORTH (WEST)	LONGITUDE WEST	REPORT DATE DAY   MO   YR
13	13	01	PA	FORBES MOUNTAIN RESERVOIR DAM	42°17'.6	72°53'.6	15 NOV 79
(2) REGION/BASIN				(3) NAME OF ENVIRONMENT			
LITTLE RIVER				CABLE MOUNTAIN RESERVOIR			
(4) RIVER OR STREAM				(5) NEAREST DOWNSTREAM CITY - TOWN - VILLAGE			
LITTLE RIVER				ESTFIELD			
(6) TYPE OF DAM				(7) HYDROLOGIC CAPACITIES			
CONCRETE				STRUCTURE HEIGHT FEET	HYDRAULIC HEAD FEET	DISCHARGE CFS	DIST FROM MILE
1931				262	250	9000	7000
CONCRETE				INITIAL HEIGHT FEET	MAXIMUM HEAD FEET	NED	N
1931				0	0	N	N
(8) REMARKS							
1970 PRELIMINARY							
(9) SPILLWAY				(10) POWER CAPACITY			
MAS. (EQUIL. HEAD) 730				MAXIMUM DISCHARGE (CFPS)	VOLUME OF DAM (CY)	INSTALLED POWER (KWH)	NET HEAD AT MAX. FEET
135				20000	1700166	33.0	0
(11) OWNER				(12) CONSTRUCTION BY			
SPRINGFIELD WATER DEPT				HAZEN WHIPPLE			
(13) DESIGN				(14) ENGINEERING BY			
CITY OF SPRINGFIELD				CITY OF SPRINGFIELD			
(15) INSPECTION BY				(16) AUTHORITY FOR INSPECTION			
IS-6 • HODD DIV SCI				INSPECTION DATE DAY   MO   YR			
15 NOV 79				15 NOV 79 PUBLIC LAW 92-367			
(17) REMARKS							
ACC 33000 CIVIC 5270 EAST 1399003							

ATE  
MED